

The Performance of House Foundations in the Canterbury Earthquakes

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Abstract

The Canterbury Earthquakes of 2010-2011, in particular the 4th September 2010 Darfield earthquake and the 22nd February 2011 Christchurch earthquake, produced severe and widespread liquefaction in Christchurch and surrounding areas. The scale of the liquefaction was unprecedented, and caused extensive damage to a variety of man-made structures, including residential houses. Around 20,000 residential houses suffered serious damage as a direct result of the effects of liquefaction, and this resulted in approximately 7000 houses in the worst-hit areas being abandoned.

Despite the good performance of light timber-framed houses under the inertial loads of the earthquake, these structures could not withstand the large loads and deformations associated with liquefaction, resulting in significant damage. The key structural component of houses subjected to liquefaction effects was found to be their foundations, as these are in direct contact with the ground. The performance of house foundations directly influenced the performance of the structure as a whole. Because of this, and due to the lack of research in this area, it was decided to investigate the performance of houses and in particular their foundations when subjected to the effects of liquefaction.

The data from the inspections of approximately 500 houses conducted by a University of Canterbury summer research team following the 4th September 2010 earthquake in the worst-hit areas of Christchurch were analysed to determine the general performance of residential houses when subjected to high liquefaction loads. This was followed by the detailed inspection of around 170 houses with four different foundation types common to Christchurch and New Zealand: Concrete perimeter with short piers constructed to NZS3604, concrete slab-on-grade also to NZS3604, RibRaft slabs designed by Firth Industries and driven pile foundations. With a focus on foundations, floor levels and slopes were measured, and the damage to all areas of the house and property were recorded. Seven invasive inspections were also conducted on houses being demolished, to examine in more detail the deformation modes and the causes of damage in severely affected houses. The simplified modelling of concrete perimeter sections subjected to a variety of liquefaction-related scenarios was also performed, to examine the comparative performance of foundations built in different periods, and the loads generated under various bearing loss and lateral spreading cases.

It was found that the level of foundation damage is directly related to the level of liquefaction experienced, and that foundation damage and liquefaction severity in turn influence the performance of the superstructure. Concrete perimeter foundations were found to have performed most poorly, suffering high local floor slopes and being likely to require foundation repairs even when liquefaction was low enough that no surface ejecta was seen. This was due to their weak, flexible foundation structure, which cannot withstand liquefaction loads without deforming. The vulnerability of concrete

perimeter foundations was confirmed through modelling. Slab-on-grade foundations performed better, and were unlikely to require repairs at low levels of liquefaction. Ribraft and piled foundations performed the best, with repairs unlikely up to moderate levels of liquefaction. However, all foundation types were susceptible to significant damage at higher levels of liquefaction, with maximum differential settlements of 474mm, 202mm, 182mm and 250mm found for concrete perimeter, slab-on-grade, ribraft and piled foundations respectively when subjected to significant lateral spreading, the most severe loading scenario caused by liquefaction.

It was found through the analysis of the data that the type of exterior wall cladding, either heavy or light, and the number of storeys, did not affect the performance of foundations. This was also shown through modelling for concrete perimeter foundations, and is due to the increased foundation strengths provided for heavily cladded and two-storey houses. Heavy roof claddings were found to increase the demands on foundations, worsening their performance. Pre-1930 concrete perimeter foundations were also found to be very vulnerable to damage under liquefaction loads, due to their weak and brittle construction.

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Important notice

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1 Introduction

1.1 Research Motivation

The 2010-2011 Canterbury earthquake sequence caused major damage to the natural and built environment in Christchurch and wider Canterbury. The earthquake sequence included the M_w 7.1 4th September 2010 Darfield earthquake, the M_w 6.2 22nd February 2011 Christchurch earthquake and a number of significant aftershocks with moment magnitudes of M_w 4.0 to 6.0. Maximum peak ground accelerations between 0.25-0.45g were recorded in central parts of Christchurch during the 22nd February 2011 earthquake.

The most significant feature of the Canterbury earthquakes was the substantial and widespread liquefaction that occurred in multiple seismic events. Due to poor soil conditions, a number of the larger aftershocks produced liquefaction in parts of the city, but the September 2010 and February 2011 earthquakes produced the largest levels of liquefaction within Christchurch. This liquefaction occurred in varying degrees across whole suburbs of the city, with the worst liquefaction occurring in the 22nd February 2011 earthquake, due to its proximity to the city and the higher seismic demands it produced. The effects of liquefaction in these earthquakes, such as loss of bearing capacity, the ejection of sand and water, lateral spreading and global ground settlements, caused extensive damage to residential houses.

The liquefaction from the 22nd February 2011 earthquake severely affected around 20,000 residential houses in Christchurch, and has since led to the abandonment of approximately 7000 residential houses in the 'red zone' along the Avon river on the Eastern side of Christchurch. Although residential houses were found to have performed well under inertial seismic loading in the earthquakes, they suffered serious damage when subjected to the effects of liquefaction. This is primarily due to the weak, flexible foundation systems that are used in residential buildings in New Zealand, which are not designed specifically to withstand liquefaction, and cannot accommodate the large loads and permanent deformations associated with it.

The performance of foundations under liquefaction loads is crucial to the performance of the overall structure, as damage to the foundation is transferred directly to the superstructure. There has been little to no research on the performance of residential house foundations experiencing liquefaction, and as the Canterbury earthquakes have shown this to be such an important issue, while also providing an extensive resource of data, the following research aims were developed to explore the performance of residential house foundations in the Canterbury earthquakes:

- How does liquefaction affect the foundations of residential houses, and how does this translate to the overall performance of houses?
- What is the correlation between the severity of liquefaction and the damage to residential house foundations?
- What are the typical types of damage and deformations caused in residential houses and their foundations when subjected to liquefaction?
- How does the performance of standard residential foundation types used in New Zealand, including the two most common, compare to each other when subjected to liquefaction?

To achieve these aims, a range of house inspections were conducted to observe and measure damage. These included 500 general inspections conducted by University of Canterbury teams following the 4th September 2010 Darfield earthquake and 170 detailed inspections conducted by the author following the 22nd February 2011 Christchurch earthquake. Special focus was given to foundations in the detailed inspections, with foundation damage quantified by recording floor levels and slopes in each house, which were used to develop foundation damage indices. An additional seven houses were inspected invasively during their demolition to better understand the deformation and damage caused by liquefaction. Simplified modelling was then carried out to understand the mechanisms involved and give an idea of the demands when foundations are subjected to bearing loss and lateral spreading and how these demands relate to the foundation capacities.

1.2 Thesis Outline

This thesis is organised into five main chapters, the contents of which are outlined here. Chapter 2 explores previous studies with regard to the manifestation of liquefaction and its effects on residential houses. Current methods for determining the triggering of liquefaction and for modelling the resulting consequences such as lateral spreading and settlement are examined. Previous research on low-rise, shallow foundations subjected to liquefaction is also investigated, along with methods currently available for use with residential structures. A review of the relevant New Zealand design standards and the current state of practice in the field is also presented.

Chapter 3 investigates the general performance of residential houses and their foundations when subjected to high levels of liquefaction and lateral spreading. This was done by analysing the data from approximately 500 house inspections that were conducted by a University of Canterbury Summer Research Team following the 4th September 2010 earthquake, in the areas of Christchurch worst-hit by liquefaction. These inspections recorded damage to the land and structure with qualitative measures. The typical types and levels of damage observed in houses subjected to liquefaction are shown, and trends in the foundation and superstructure performance are examined, along with other construction features such as house age and weight.

The performance of four residential house foundation types common in New Zealand is analysed in Chapter 4. These are concrete perimeter with short pier supports constructed to NZS3604, concrete slab-on-grade foundations constructed to NZS3604, the proprietary RibRaft™ slab foundation system of Firth Industries, and driven pile foundations. Approximately 170 houses, divided evenly between the four foundation types, were inspected in detail by the author following the 22nd February 2011 earthquake. The inspections were conducted across the city, encompassing a full range of liquefaction severities, in order to examine the performance of the different foundation types under a range of liquefaction levels. They included 19 inspections of houses experiencing no liquefaction to provide a level of comparison. The typical deformation modes of each foundation type are outlined, and the overall performance of superstructures and foundations, along with the comparative performance of each foundation type is evaluated. This evaluation is based on a variety of data indices which were collected during the inspections. Foundation damage, represented by measured floor levels and slopes, is compared against ground damage and superstructure damage ratings that were based on qualitative criteria. Trends in other structural details such as cladding type, age of construction and number of storeys are also investigated.

Chapter 5 explores in detail the deformation characteristics and damage caused in concrete perimeter and slab-on-grade foundation houses subjected to severe land damage. This uses the results of seven houses which were invasively inspected by the author during their demolition. For each house, the cause of any deformation and damage is scrutinised, and related to the ground conditions and foundation details which were determined during the investigations.

Chapter 6 analyses the mechanisms and loading demands caused by liquefaction related bearing loss and lateral spreading under a concrete perimeter foundation. The moment demands from three different bearing capacity loss models were obtained with the simplified elastic analysis of a concrete perimeter beam supported by soil springs. The springs were degraded in stiffness by varying degrees, and over a range of lengths, to represent a variety of liquefaction severities with bearing loss under the centre of the concrete perimeter beam, at one end, and at both ends. The drag force caused by lateral spreading along a concrete perimeter beam was also calculated for a range of lengths. The calculated demands are compared against the capacities of eight different concrete perimeter sections that represent different periods of construction, to better understand their performance in liquefaction related loading scenarios. Conclusions are drawn on the influence of the type of loading and the weight of the house on the demands produced and how the age of foundation construction influences performance.

Final conclusions, recommendations and future research are included at the end of the thesis, with additional information included in appendices.

2 Background

2.1 The Canterbury Earthquakes

From September 2010 to the end of 2011, Christchurch and the Canterbury region experienced a number of significant seismic events that caused substantial and widespread damage to structures, services and land across the region. Arguably the most significant aspects of these earthquakes were the geotechnical (Wood et. al. (2010), Allen et. al. (2010) and Cubrinovski et. al. (2011)), namely the liquefaction and lateral spreading that occurred over large areas of the region. This caused the majority of damage to natural environment and man-made structures in terms of level, extent and cost of damage. In particular, the majority of damage to residential houses, which stood up very well to the inertial loading of the seismic shaking was caused by the liquefaction and lateral spreading.

The major seismic events during this sequence are shown in Table 2.1. The most significant events in terms of the geotechnical damage caused were the 4th September 2010 and 22nd February 2011 earthquakes. These caused the most severe and widespread liquefaction and lateral spreading throughout the region, consequently causing the most damage to man-made structures. The 13th June 2011 and 23rd December 2011 aftershocks also caused significant liquefaction and lateral spreading in some parts of the city. The other events listed in Table 2.1 also caused smaller amounts of liquefaction in isolated areas.

Table 2.1. Characteristics of the main seismic events during the 2010/2011 Canterbury Earthquake Sequence (Bradley (2013) and Canterbury Geotechnical Database).

Date	M _w	Distance from Christchurch CBD	Depth	PGA in the CBD***
4 th September 2010	7.1	40km West	10km	0.17
19 th October 2010	4.8	<10km South	5km	0.09
26 th December 2010	4.7	<10km South	5km	0.16
22 nd February 2011	6.2	<10km South	6km	0.37
16 th April 2011	5.0	10km Southeast	9km	0.15
13 th June 2011*	6.0 & 5.3	10km East	7km	0.22
23 rd December 2011**	5.8 & 5.9	10km East	7km	0.22

*Two events 80 minutes apart. **Two events 20 minutes apart.

***From the CHHC strong ground motion site

Figure 2.1 and Figure 2.2 show the areas affected by liquefaction during the 4th September 2010 and 22nd February 2011 earthquakes. It is obvious from comparison between the two figures that the 22nd February 2011 earthquake caused much more widespread liquefaction in Christchurch than the 4th September 2010 event. On the whole the liquefaction was also more severe in each area it occurred

when compared to the 4th September 2010 event. This was because the February earthquake occurred much closer to the city than the September earthquake (Table 2.1), producing much larger peak ground accelerations and cyclic stress ratios as a result.

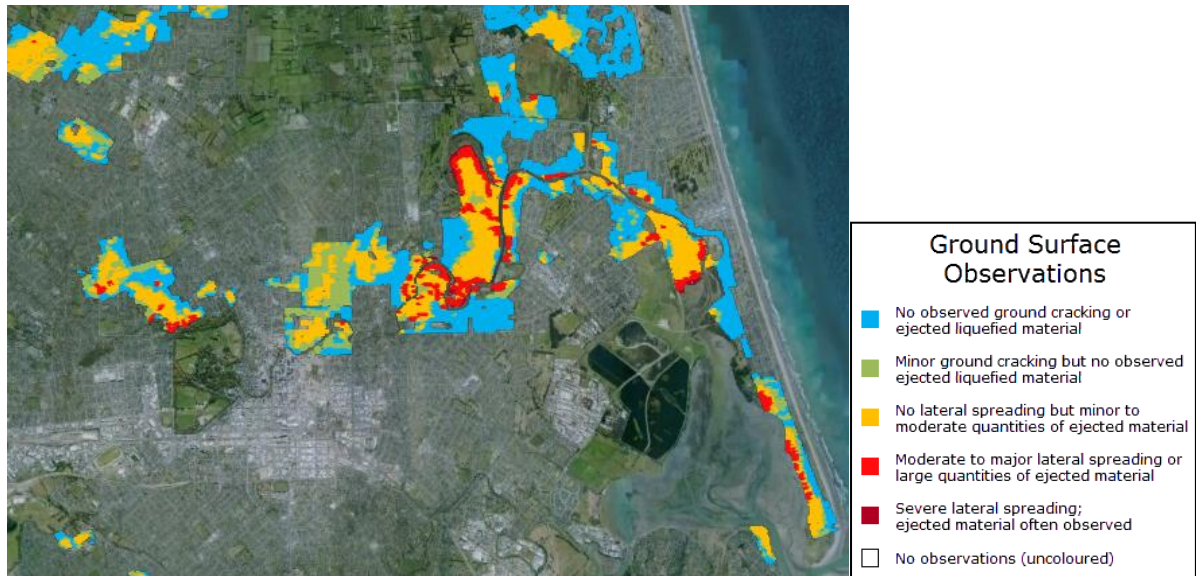


Figure 2.1. Areas of liquefaction (in Christchurch) in the 4th September 2010 earthquake (Map Layer CGD0300, Canterbury Geotechnical Database (2013)).

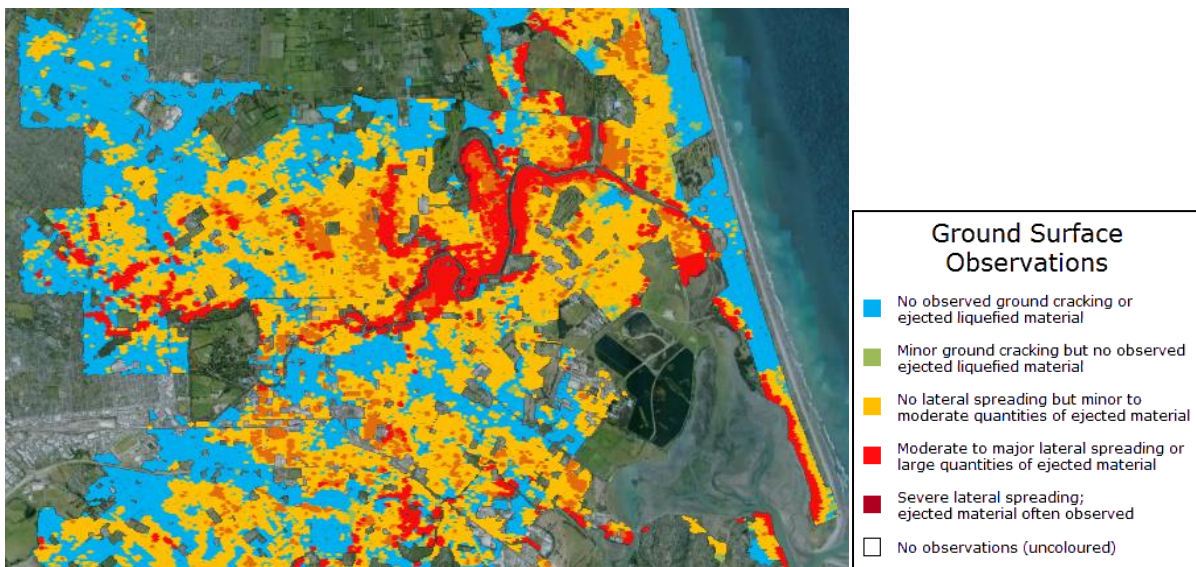


Figure 2.2. Areas of liquefaction (in Christchurch) in the 22nd February 2011 earthquake (Map Layer CGD0300, Canterbury Geotechnical Database (2013)).

2.2 Liquefaction

Cyclic Liquefaction occurs when saturated, cohesionless soils are subjected to a sufficiently large level of cyclic stress for a sufficient period of time (i.e. a certain number of cycles). When a drained soil is cyclically loaded (i.e. in an earthquake), it has a general tendency to contract, reducing in volume, as the soil is sheared and particles are rearranged closer together. However, if the soil is undrained, (as is regularly the case during an earthquake, due to the short-term duration of the shaking and relatively small permeability of soils) the soil cannot immediately contract as it would when

sheared under drained conditions. This causes the development of excess pore water pressure, u_E . The excess pore water pressure increases as the shaking continues, which in turn reduces the effective stress in the soil skeleton, $\sigma'_v = \sigma_v - (u + u_E)$. If the earthquake produces large enough stresses, that continue for a sufficient period of time, the zero effective stress condition can be eventually reached, where $u_E = \sigma'_{v0}$ and $\sigma_v - (u + u_E) = \sigma'_v = 0$. At this point, the soil is said to be liquefied, with zero strength, and practically acts as a viscous fluid. This process of excess pore water pressure build-up is illustrated in Figure 2.3 from Cubrinovski (2010). During liquefaction, the high pore water pressures cause water to flow up to the surface, carrying sand and silt material with it. When it is able to break the surface, this pore water produces flooding and 'sand boils', the characteristic surface manifestation of liquefaction, shown in Figure 2.4.

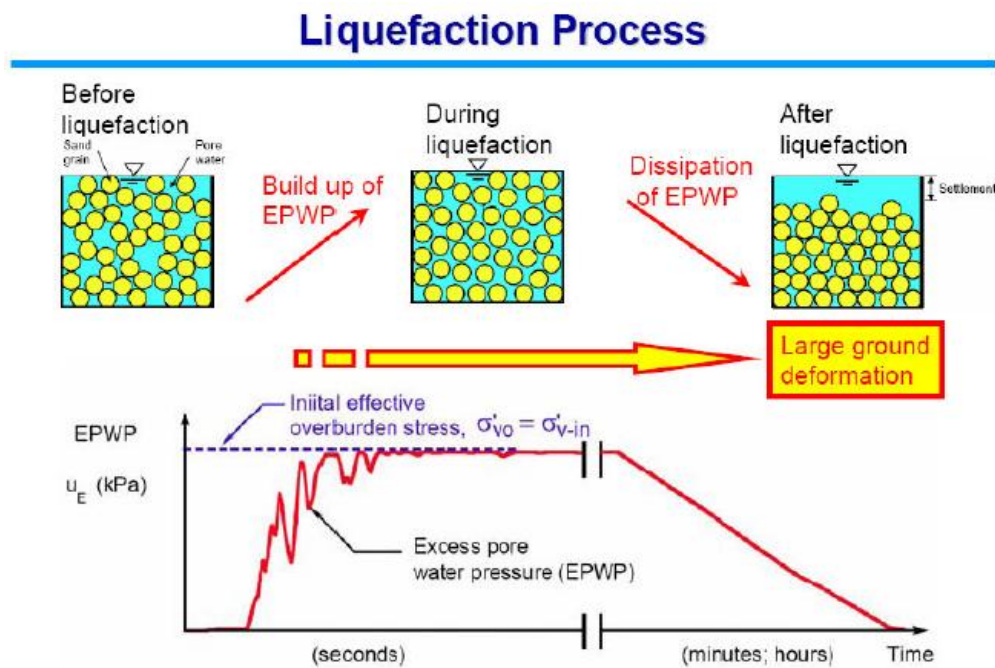


Figure 2.3. Schematic of the build-up of excess pore water pressure and creation of liquefaction (Cubrinovski, 2010).



Figure 2.4. A sand boil on the ground surface, caused by liquefaction at depth.

There are a wide range of soils known to be susceptible to liquefaction, with a number of criteria for soil to be classified as liquefiable. First of all, a soil has to be saturated to allow the generation of excess pore water pressures. While there is the possibility of soil above the water table liquefying due to the upward flow of water from liquefied soils below, the largest effect is caused by the soils below the water table. Second, the soil must be non-plastic. Clays and plastic silts have been shown not to liquefy, however, most other soils can liquefy. Anything from non-plastic silts to silty sands to gravels can liquefy under the right conditions. The third factor that must be considered is the density of the soil in question. In the simplified procedure of Seed & Idriss (1982), soils with a Standard Penetration Test (SPT) N-value of more than 30 are considered non-liquefiable, as they are too dense for excess pore water pressures to push the soil particles apart.

Liquefaction has a range of consequences for the natural and built environment. The typical surface manifestation of sand and water ejecta can cause flooding of low lying areas including streets and houses. When the excess pore pressure dissipates following the earthquake, settlement occurs as the particles re-consolidate. This settlement can be global or differential due to locally differing soil conditions and non-uniformity of liquefaction manifestation.

There is evidence to suggest that the presence of a crust layer on top of liquefied soil may reduce the surface effects of liquefaction (Karamitros et. al. 2011). While it is unlikely to reduce the susceptibility of any saturated underlying layer to liquefaction, the crust can provide a stabilising effect to structures above ground. Any soil profile with a water table below the ground surface effectively has a crust of non-liquefiable soils equal in thickness to the depth of the water table. This may extend below the water table if there are dense or cohesive non-liquefiable soils present. The soil crust layer acts as a stabilising layer, reducing differential settlements at the surface, and providing some bearing capacity for overlying structures. If the crust is dense or made of cohesive soils it can also prevent the surface manifestation of liquefied soils underneath, preventing water and sand ejecta coming to the surface.

Another major consequence of liquefaction is lateral spreading. Lateral spreading occurs in either sloping ground or where a free-face is present, such as a river bank or quay wall. These geometric features create a static shear bias in the soil. During an earthquake this is combined with the inertial forces of the soil due to cyclic shaking. When soil at depth is liquefied, the cyclic loading causes gradual movement of the soil down the slope, or towards the free face. This large-scale movement of soil can extend for hundreds of metres from a free face, with total movement at the free face of up to several meters. The movement causes cracks to open up in the ground, which can be anything from a few millimetres to metres wide and deep.

2.2.1 Impacts of Liquefaction on Engineering Structures

Earthquake-induced liquefaction and its associated effects can have an influence on engineering structures in a number of ways. The sand and water ejecta can cause flooding of low lying areas including streets and houses. This can cause cosmetic and structural damage to structures which can be damaged by flooding and sand inundation. Road surfaces are often broken up and distorted by the ejecta surfacing, creating humps and large potholes which are then obscured by the flooding water. Houses, particularly those with unreinforced concrete slab foundations, can have their foundations crack, allowing sand and water into the house, damaging floor coverings and electrical items.

When liquefaction occurs, the soil is effectively a liquid. Light structures buried within the soil, such as manholes, can float up and break the surface. This also causes distortion and damage to the pipelines they are connected to. Heavy structures such as buildings founded on top of the liquefied deposit can sink into the soil, resulting in global settlements which damage exterior service connections. Differential settlements are also common, due to irregular structural loads and soil profiles, which can result in major structural damage and tilting. These damages can require difficult and costly repairs, as whole structures have to be re-leveled and their foundations repaired or rebuilt. Broken pipelines such as water, stormwater and sewerage can become clogged with the sand ejecta, making for a long and costly repair process.

Lateral spreading often causes the most extreme damage to structures. Pipelines can be torn apart and offset longitudinally or transversely, or crushed together due to the opening up of the ground. Bridges are regularly damaged by lateral spreading, as it is common close to rivers. Abutments are pushed towards each other, which can cause tilting and damage to the abutments, and compressional buckling of the superstructure. Buildings, particularly houses, located within the lateral spreading zone are also severely damaged. Cracking in the ground can go through houses, cracking and separating foundations in tension and shear, which in turn creates large cracks and zones of damage through the superstructure. Foundations can also be separated from underneath buildings if not properly attached due to the differential nature of the movement. The nature of lateral spreading also means that ground slopes change locally, as different sections subside more or less during spreading, creating large tilts and differential settlements on structures.

2.3 Impacts of Liquefaction on Residential Houses

As has been documented by a number of sources, residential houses of the style seen in Christchurch and New Zealand (predominantly light timber-framed one and two-storey buildings) located on the flat resisted the seismic shaking during the Canterbury earthquakes very well (Buchanan & Newcombe 2011, Beattie et. al. 2011, Buchanan et. al. 2011). This is due to the characteristics of their construction as strong and flexible structures with a large degree of redundancy and plenty of lateral

strength and load-dissipating mechanisms. However, their light and flexible nature means they are easily damaged by liquefaction and its effects in a number of ways.

When soil liquefies, it instantly loses most of its bearing capacity. This results in settlement of structures founded on top of the liquefied soils. This either occurs globally, as the system including ground and structure settles both during liquefaction under the weight of the structure during the earthquake and then as the soil re-consolidates and compacts during excess pore water dissipation after the event, or locally, where a combination of the weakness of the crust layer, and the weight of the structure causing large bearing pressures can cause the structure to punch through the crust layer, settling more than the surrounding ground. Liquefaction is a relatively rapid and highly varied process. When combined with the highly variable ground conditions in Christchurch, and differential loading found in any structure, liquefaction leads to complex settlements, which are often differential in nature and difficult to predict (Figure 2.5).

The powerful ejection of sand and water during liquefaction can also damage residential houses, which are characterised by weak, flexible foundations not generally designed to withstand the forces produced by liquefaction. The ejecta can cause uplift of the foundations in localised areas, which when combined with overall settlements can produce large stresses within the foundation system, leading to cracking, separation and permanent displacements.

Lateral spreading also has a severe effect on houses. It causes large differential settlements of the ground as it slumps towards a free face, and also allows cracks to open in the ground. These displacements are transferred to light, flexible houses on shallow foundations that have very limited capacity to resist the large loads caused by the displacements (Figure 2.5).

The residential timber-framed houses used in New Zealand have excellent resistance to the inertial loading imposed during an earthquake, as their superstructures are strong and flexible, with a large degree of redundancy and many load-dissipating systems (Beattie et. al. 2011). Because the superstructures are light relative to their strength and flexibility, the inertial forces can be accommodated by drift without serious damage.

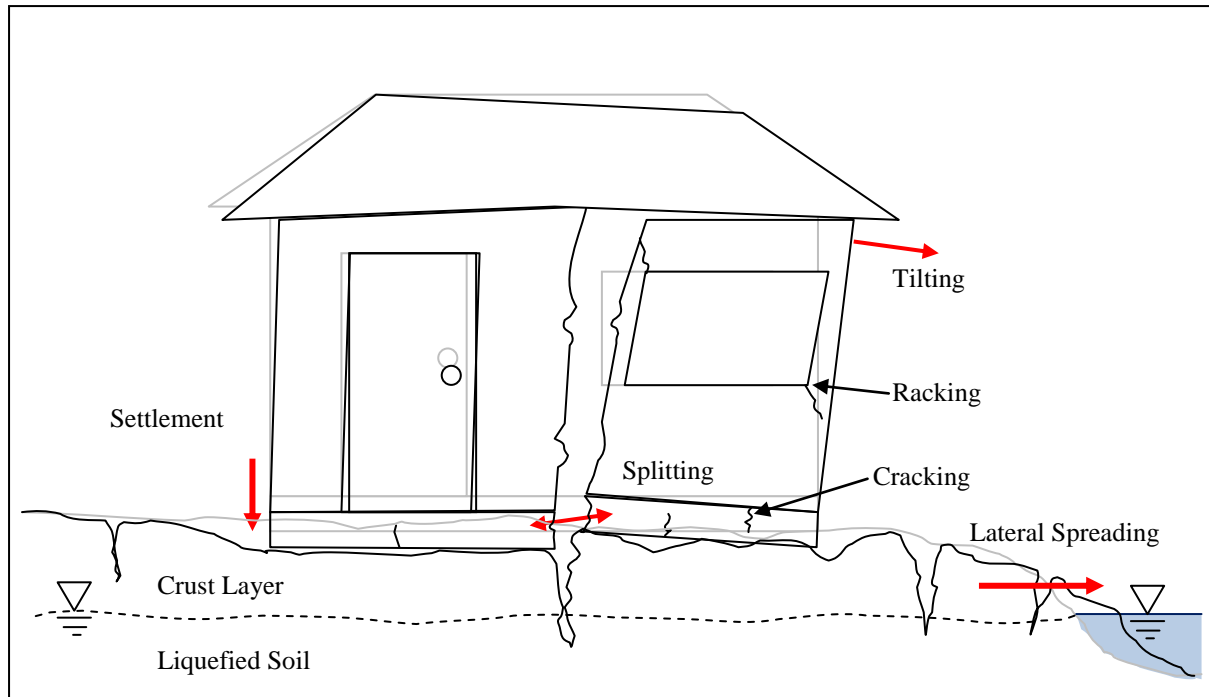


Figure 2.5. Schematic illustration of the effects of liquefaction on a typical residential house due to loss of bearing capacity and lateral spreading.

It is quite a different story when it comes to ground loadings caused by liquefaction. Compared to the inertial loads on a small, light structure such as a residential house, ground loads are very large and not proportional to the size or weight of the structure. These large loads are caused by the displacements imposed on the ground by liquefaction, both transient (during the earthquake) and permanent (after the earthquake). Although the superstructures of residential houses are flexible, they are not flexible enough to deal with these large displacement demands. Damage to houses caused by these displacements is generally visible in the stiffer more brittle materials used in house construction, such as claddings, both interior plasterboards and exterior brick, concrete block or stucco, and also the concrete used in foundations. When these materials are subjected to ground displacements, they will crack. Additional structural deformation may concentrate at cracked sections of foundations, walls and floor supports, leading to openings and localised deformation.

2.3.1 Observations from the Canterbury Earthquakes

A number of articles have been published on the performance of various aspects of houses in the Canterbury earthquakes, based on reconnaissance and inspections following the two main earthquakes of the 4th September 2010 and the 22nd February 2011. Allen et. al. (2010) and Cubrinovski et. al. (2011) investigated the geotechnical aspects of the two earthquakes in relation to (among other things) residential structures. Buchanan & Newcombe (2010) and Buchanan et. al. (2011) investigated the overall performance of houses in each earthquake, and Beattie et. al. (2011), Beattie & Liu (2012), Thomas & Shelton (2012) and Thomas et. al. (2013) studied various aspects of the performance of houses not subject to liquefaction.

2.3.1.1 Shaking Damage

Buchanan & Newcombe (2010), Buchanan et. al. (2011) and Beattie et. al. (2011) found that by and large, residential houses on the flat, which were not subject to liquefaction performed very well, and no foundation failures were seen amongst these houses (Beattie et. al. 2011). Very few windows were broken, indicating low drift levels despite the high level of shaking (Buchanan & Newcombe 2010). Typical shaking damage to houses observed was the cracking of internal claddings such as gypsum plasterboard and other, older wall linings. These cracks were often located at the corners of wall openings such as window and door frames. They were either vertical, opening up at the interface between plasterboard panels, or occasionally diagonal where the plasterboard had been cut around the wall openings. Despite the cracking, in most cases the plasterboard retained its structural integrity and was still able to resist lateral load as it is designed to do, all-be-it with a small amount of nail-slip required to be engaged before it could provide lateral resistance. Heavy exterior claddings, consisting of variations on brick or stone masonry or concrete block construction often suffered minor damage, seen as cracking along joints. In a small number of cases there was complete or partial separation and/or collapse of these claddings, however in most cases these problems were caused by factors such as poor construction (Beattie et. al. 2011). There was very little to no visible shaking damage to weatherboard claddings, as their flexible nature and ductile behaviour as a system means they do not rupture, but resist deformation through nail slip and joint movement.

There was little to no shaking damage to ceilings or roof framing in either earthquake, except where brick chimneys were dislodged and fell through the roof, or in some areas on the Port Hills, where extreme horizontal and vertical accelerations caused heavy roof tiles to be thrown off houses (Thomas et. al. 2013).

It was found that buildings with heavy roof and wall claddings suffered higher levels of structural and cladding damage, which is attributed to the greater seismic mass involved in excitation (Buchanan & Newcombe 2010, Buchanan et. al. 2011). It was also found that houses with highly irregular floor plans (such as an “L” or “U” shape) often suffered additional structural damage, however this damage was generally confined to the intersections between wings (Beattie et. al. 2011).

It has been suggested that the improvement in building codes over the years has offset the move amongst new home builders towards more open-plan and structurally irregular buildings which are more easily damaged. Older codes were less prescribed, and older houses often were more poorly constructed, but their layouts as regular-in-plan with segmented rooms led to stable structures with a much greater degree of redundancy in their load paths than modern houses (Beattie et. al. 2011). The large degree of irregularity in stiffness and strength both vertically and in plan of new houses, which often have large openings and open-plan areas to the North for sun or towards views, causing

irregularities in the lateral and torsional load-resisting elements, can lead to higher levels of damage than more regularly constructed houses (Liu & Beattie 2012 and Buchanan et. al. 2011).

2.3.2 Liquefaction Damage

Liquefaction related damage to residential houses in the Canterbury Earthquakes was much more serious than the damage in areas that didn't experience any liquefaction. As shown in Figure 2.1 and Figure 2.2, the areas of liquefaction within Christchurch were widespread in both major earthquakes. Some of the areas that suffered more severe liquefaction effects also liquefied again in many of the large aftershocks shown in Table 2.1. The scale and extent of the liquefaction seen in the Canterbury Earthquake sequence is likely to be very rare (Cubrinovski et. al., 2011). It provides a valuable source of data on the performance of houses subjected to liquefaction effects, unprecedented both in quality and quantity. Approximately 20,000 residential houses were severely affected by liquefaction in the 2010-2011 Canterbury earthquakes. The severity and widespread nature of this liquefaction has led to the abandonment around 7000 houses in the worst-hit areas of Christchurch – predominantly in the Eastern suburbs close to the Avon river, known collectively as the residential 'Red Zone', which is shown in Figure 2.6 (New Zealand Government, 2011).

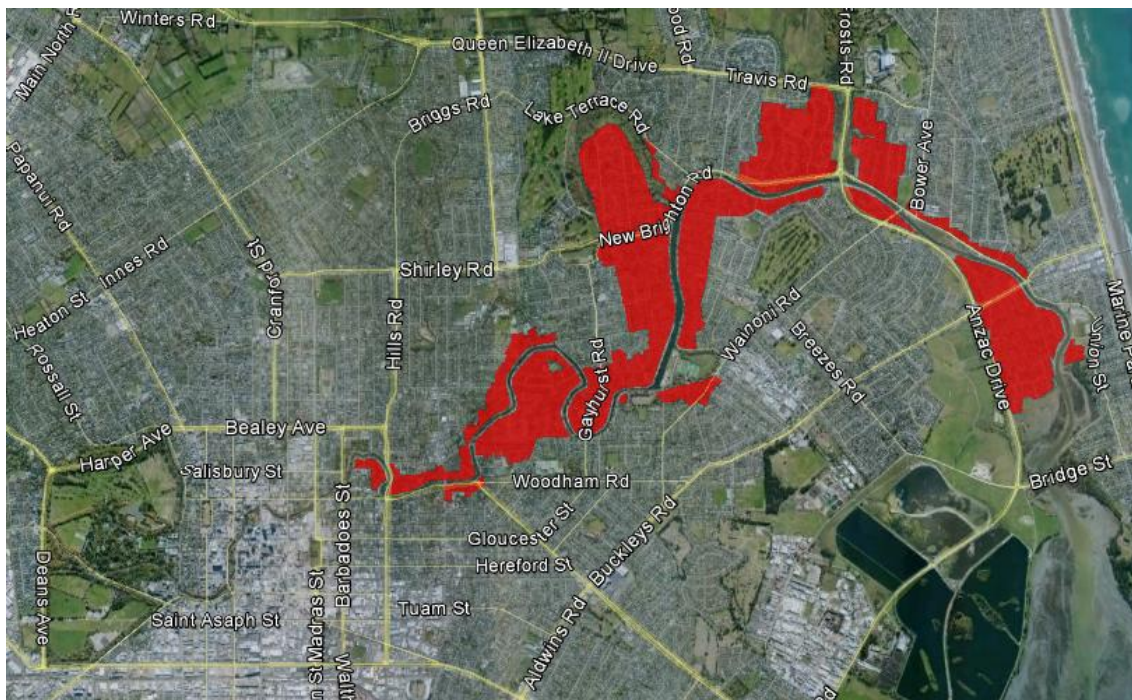


Figure 2.6. Residential 'Red Zone' in Christchurch following the 22nd February Earthquake.

Buchanan & Newcombe (2010) and Buchanan et. al. (2011) found a range of damage in residential buildings after the two main earthquakes, which depended on the foundation type, structural form and cladding materials used.

Following the 4th September 2010 earthquake, Buchanan & Newcombe (2010) observed that in general, buildings which experienced uniform settlement maintained their structural integrity.

However, buildings which settled differentially suffered large levels of damage to their foundations, external claddings and internal linings. Heavier buildings seemed to suffer greater damage due to higher levels of settlement. After the 22nd February 2011 earthquake, Buchanan et. al. (2011) found that heavier claddings (such as brick and concrete block) were more heavily damaged. This was often due to poor connections between elements (i.e. the mortar in the brickwork or the brick-to-framing connection). But it was also affected by the different structural stiffnesses of the elements (i.e. flexible timber framing joined to stiff brickwork). Weatherboard cladding behaved very well and exhibited much less damage. It was noted that severe damage to houses was often not the result of structural collapse, but rather due to a lack of functionality with doors and windows not opening or closing, and non-structural elements that provide weather and sound resistance being badly damaged and no longer able to fulfil their role.

Various foundation systems were also seen to perform differently. Buchanan et. al. (2011) observed that concrete perimeter foundations with short piers performed well, as the continuous foundation wall provided good bearing capacity and held the house together and the stiff floor plate transferred forces to the perimeter. However the extreme loads of lateral spreading under these foundations caused the foundation walls to crack and pull away from the timber-framed floor system. Slab-on-grade floors constructed to NZS3604 were found to have cracked under lateral spreading and settlement loads as they do not have sufficient stiffness and strength to bridge severe land deformations beneath the slab. Some flexural cracks were found which were caused by a loss of bearing capacity or settlement of the ground underneath the foundation. This slab cracking, particularly due to lateral spreading, caused superstructure framing damage, localised around the location of the slab rupture. The failure of slab-to-foundation wall connections was also seen in a number of cases. It was found that specialist slab foundations that were designed to be better than NZS3604, such as RibRaftTM (Firth, 2003) and slab on piles performed better than standard slab-on-grade floors due to added stiffness and support. However they were still not strong enough to resist all ground deformations.

2.4 Modelling of Liquefaction Effects

There are a number of considerations that must be taken into account when attempting to predict the effects of liquefaction on structures, such as residential buildings. The process begins with calculations to determine whether the soil is susceptible to liquefaction and lateral spreading, and is followed by the calculation of settlements, both in the free field and for structures.

2.4.1 Triggering

The first step is to determine whether the underlying soils are susceptible to liquefaction, and if liquefaction will be triggered during a particular earthquake. There has been much research in this field over the last 40 years, and methods for determining triggering are well established.

The most common method used is the simplified procedure developed by Seed and Idriss (1982), with refinements by Youd and Idriss (2001) and Idriss and Boulanger (2006). This uses a lower bound factor of safety approach, defined as

$$FS = CRR/CSR_{7.5}$$

Where $CSR_{7.5}$ is the Cyclic Stress Ratio caused by the earthquake, defined as

$$(CSR)_{7.5} = \frac{CSR}{MSF} = 0.65 \left(\frac{\sigma_{v0}}{\sigma'_{v0}} \right) a_{max} \frac{r_d}{MSF}$$

Which varies with depth by the stress ratio σ_{v0}/σ'_{v0} and the reduction factor r_d for which various approximations have been suggested. $CSR_{7.5}$ is a function of the Peak Ground Acceleration, a_{max} and includes a Magnitude Scaling Factor, MSF to account for magnitude effects on duration and intensity of shaking. There are a number of empirical relationships for the Cyclic Resistance Ratio, CRR, of the soil, related to either the SPT(N) blowcount, CPT tip resistance q_c or shear wave velocity V_s . The SPT-based curves are shown in Figure 2.7 where liquefaction is possible for $CSR_{7.5}$ and SPT(N) states above the curves, and will not occur for states below them. This method is used widely in practice, as it is simple and based on a large amount of field data. However it has limitations in its simplicity and lack of mechanics basis. It also requires adjustments for fines-containing sands.

The other option when modelling liquefaction triggering is to use non-linear effective stress programs, such as the program developed by Cubrinovski (1998, 2011). This fully models the build-up of excess pore water pressure, and the onset of liquefaction.

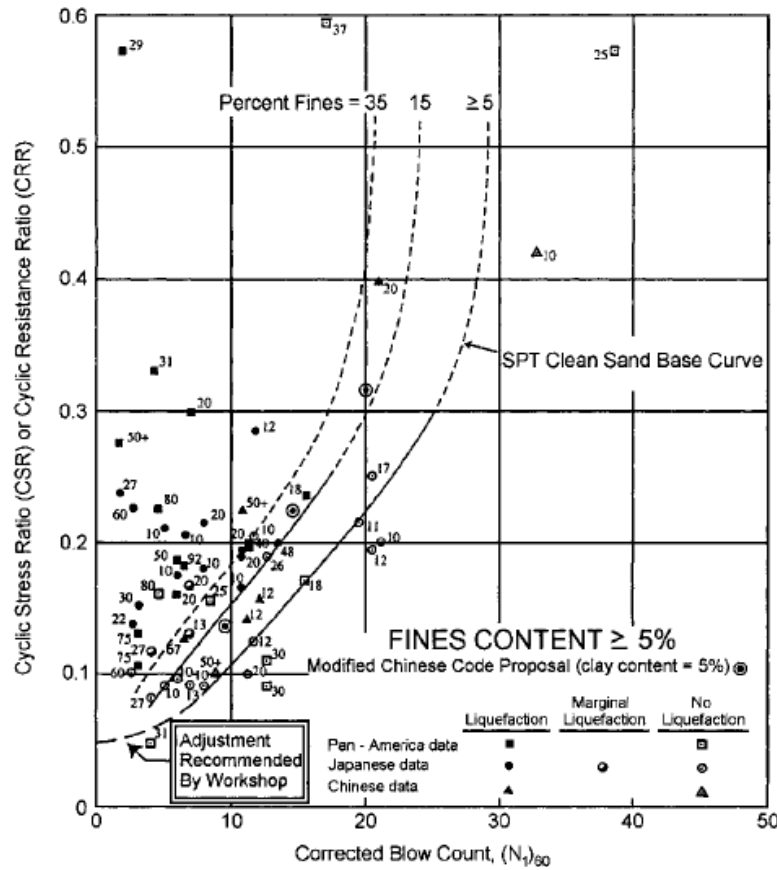


Figure 2.7. SPT clean sand based curve for Magnitude 7.5 Earthquakes with Data from Liquefaction Case Histories (Modified from Seed et. al. 1985) - Youd et. al. (2001).

2.4.2 Lateral Spreading

Bartlett and Youd (1995) proposed empirical correlations for estimating the magnitude of lateral spreading during an earthquake. They were developed by performing a multiple linear regression on a large database of case histories from the US and Japan, taking into account many different factors. Corrections were made by Youd et. al. (2002), with the current form of the equation as

$$\begin{aligned} \log D_H = & -16.713 + 1.532M \\ & - 1.406 \log R^* - 0.012R + 0.592 \log W \\ & + 0.540 \log T_{15} + 3.413 \log(100 - F_{15}) - 0.795 \log(D50_{15} + 0.1\text{mm}) \end{aligned}$$

Where:

- D_H = estimated lateral ground displacement (m)
- M = moment magnitude
- R = smallest source-to-site distance (km)
- T_{15} = cumulative thickness of saturated granular layers with $(N_1)_{60} < 15$ (m)
- F_{15} = average fines content for the granular materials within T_{15} (%)
- $D50_{15}$ = average mean grain size of granular materials within T_{15} (mm)
- S = ground slope
- W = free face ratio = height (H) / distance (L) from base of free face

This equation has a large variability, where most measured displacements fall within a range of 0.5 to 2 times that calculated with the model.

Another possibility is to use the Newmark Sliding Block approach (Newmark, 1965), where the soil, which has the potential to slide, is treated as a single degree of freedom, rigid-plastic system. Forces such as weight, uplift and friction are then equated to determine the overall movement. This approach has the disadvantage of over simplifying the actual behaviour of soil during lateral spreading, and tends to over-predict movement.

2.4.3 Settlement

The next step in modelling liquefaction effects on buildings is to calculate settlement. There are a number of methods available to calculate free-field settlement, from simplified methods based on experimental data and soil properties, mostly for sands, to full finite element analyses.

In the free field, several mechanisms contribute to settlement of the ground due to liquefaction. These include the loss of soil volume due to sand ejecta and some post-liquefaction densification, as the excess pore water pressures built up during the strong shaking dissipate, and the liquefied soil re-consolidates from its fluid state. This process can take anywhere from minutes to hours to days, with the soils remaining soft, and continuing to settle.

There are a number of procedures available for estimating the free-field post-liquefaction settlements that occur as a result of the dissipation of these excess pore water pressures. For level ground, settlement can be calculated from the volumetric reconsolidation strains induced as excess pore water pressures dissipate. The level of volumetric strain depends on the penetration resistance of the soil as well as the CSR of the earthquake.

Tokimatsu and Seed (1987) were the first to develop simplified methods for the estimation of settlement of sands subjected to earthquake loadings. These methods used laboratory data to determine curves for volumetric strain level depending on the CSR and normalised SPT $(N_1)_{60}$ value of a deposit (Figure 2.8.(a)). This work was followed by Ishihara and Yoshimine (1992) who developed Figure 2.8.(b) to determine volumetric strain based on either relative density, D_r , N_1 or q_c and a factor of safety. Wu (2002) proposed another curve which gives volumetric strain in terms of the CSR imposed and $(N_1)_{60,cs}$ of the soil. Zhang et. al. (2002) proposed similar curves using q_c rather than $(N_1)_{60,cs}$. These methods were all developed for use with sands and are not really applicable to other soils.

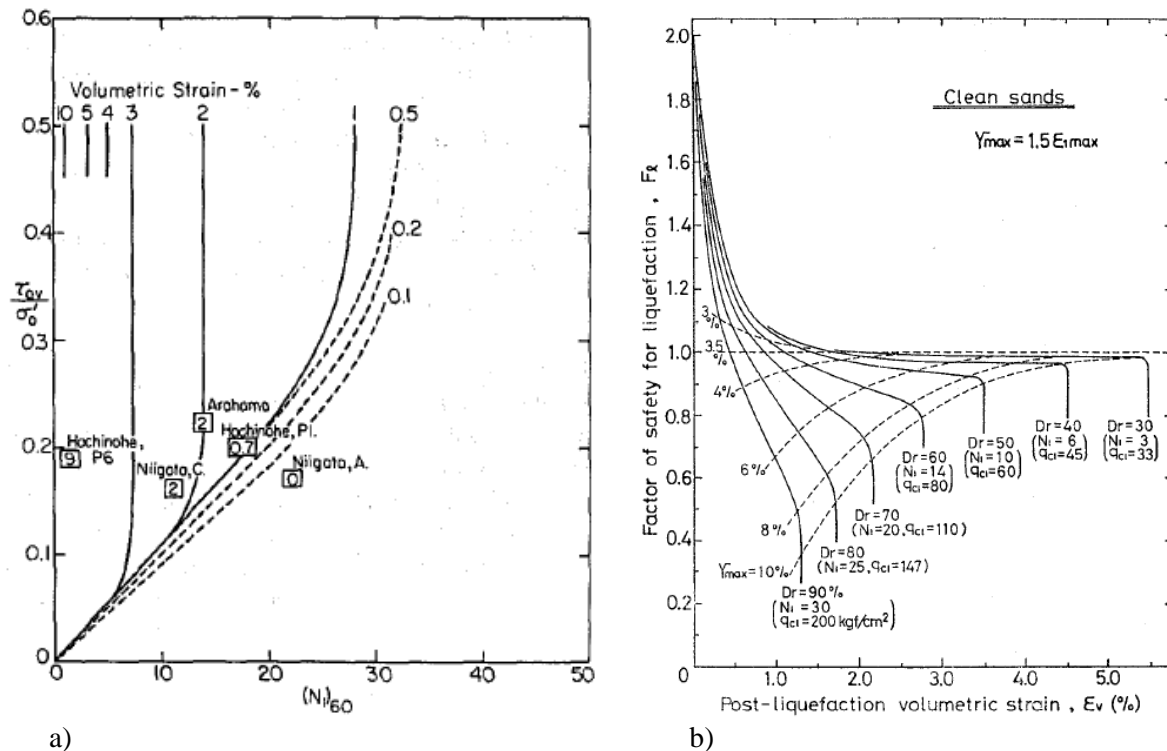


Figure 2.8. a) Relationship between Cyclic Stress Ratio, $(N_1)_{60}$, and Volumetric Strain for Saturated Clean Sands (Tokimatsu and Seed, 1987). b) Chart for determining volumetric strain as functions of factor of safety (Ishihara and Yoshimine, 1992).

There has been very little work focussed on the estimation of settlements in fine-grained soils, such as silts. These soils are also susceptible to liquefaction, as was seen in Christchurch during the Canterbury earthquakes. Sanin and Wijecwickreme (2006) tested normally consolidated Fraser River Delta silt, producing curves for volumetric strain with excess pore water pressure ratio and excess pore pressure ratio for number of cycles. However these results are limited in use to soils with properties close to those that were tested.

2.4.4 Foundations and Settlement

While there are a number of methods for determining the triggering of liquefaction, and estimating ground movements, such as free-field settlement and lateral spreading, there are currently no widely recognised, reliable methods for estimating ground performance during and after liquefaction under or close to buildings. Although there are ways to model a specific building foundation-soil system using constitutive or finite element models, this is generally only performed for large-scale, important building projects. Soil-structure interaction in these problems can be significant, but its influence is often not taken into account even for high rise buildings, let alone houses.

2.4.4.1 Research on the Settlement Characteristics of Low-Rise Shallow Foundation Structures

There is very little research on the performance and settlement characteristics of residential houses founded on liquefiable soils, and as such, it is prudent to review the existing research on larger low-

rise structures with shallow foundations, which are closer in structural form to the very light and shallow-founded residential houses in New Zealand.

The mechanisms for settlement of buildings affected by liquefaction are quite different to those in the free field. Unlike settlements in the free field, which occur post-earthquake during the re-consolidation phase, building settlements are mostly due to the failure of the underlying foundation soils, caused by a combination of the static and inertial loads from the foundation, earthquake-induced shear stresses and the shear strength degradation of the soil due to liquefaction.

A number of studies have been conducted, by Yoshimi & Tokimatsu (1977), Tokimatsu et. al. (1994), Dashti et. al. (2010) and Andrianopoulos et. al. (2005), mostly using centrifuge testing, to identify the characteristics of foundation settlement of low-rise structures with shallow foundations during liquefaction. These studies found that the contribution of post-liquefaction re-consolidation settlement to the overall building settlement was very small. The majority of settlement occurred approximately linearly with time during strong shaking, due to the different mechanisms in play, and the structures settled more than the surrounding ground. Dashti et. al. (2010) found that once the liquefied layer is thick enough, building settlements are no longer proportional to the thickness of the liquefied layer, but are due to other factors discussed below. This is unlike the free field where settlements are related to consolidation and a thicker liquefied layer almost always means larger settlements.

In Dagupan City following the 1990 Luzon Earthquake, Tokimatsu et. al. (1991) found that the confining stress due to surrounding structures in the soil under and surrounding a building was the most critical factor for the level of settlement. Buildings on corners, away from other buildings, surrounded by lightweight structures or with greater separation between buildings settled more than structures with higher confining stresses in the soil beneath. Residential houses in Christchurch almost all fall into this category, as they themselves are light structures, and are generally (relative to the size of their building footprint) far away from neighbouring structures, which are generally light buildings (other houses) anyway.

Foundation width is another key parameter for shallow foundation building settlement which was identified in the 1990 Luzon Earthquake by Tokimatsu et. al. (1994) and in research by Yoshimi & Tokimatsu (1977). Related to this were the findings of Sancio et. al. (2004) in Adapazari in the 1999 Kocaeli Earthquake, that building settlements were proportional to contact pressure of the foundation, and that the height over width (H/B) ratio of the building affected the level of tilt and the development of full bearing capacity failure. A number of studies (Yoshimi & Tokimatsu, 1977; Whitman & Lambe, 1982, 1988; Rollins & Seed, 1990; Liu 1992) have found that excess pore water pressures under foundations are lower than in the free field. This is due to the higher overall confining stress under the foundation, and can prevent liquefaction from being triggered (Andrianopoulos et. al. 2005). Dashti et. al. (2010) found that larger foundation area for the same building weight allowed larger

pore water pressures to develop during shaking, meaning liquefaction could occur more easily, whereas smaller pore water pressures were developed for a higher contact pressure. Andrianopoulos et. al. (2005) also noted that additional static shear stresses at the edge of the foundation (due to building weight) bring the soil closer to failure, so can be more easily liquefied. This may explain the presence of liquefaction ejecta at the edge of foundations, as there is effectively a non-liquefied raft beneath the foundation, due to the higher confining stresses there, causing liquefied material outside this region to be pushed up beside the foundation. This liquefaction ejecta along the perimeter of the foundation can also be caused by the settlement of the building into the ground and the ratcheting action that occurs as a result of this.

Eurocode 8 (British Standards Institution, 2006) treats liquefiable soils as extreme ground conditions. This is based on studies by Yoshimi & Tokimatsu (1977), Ishihara et. al. (1993) and Sancio et.al. (2002). As such, the use of shallow foundations is only allowed with adequate ground improvement to densify the soil and increase liquefaction resistance and/or the implementation of drainage to reduce excess pore water pressure build-up, otherwise piled foundations should be used. This has led to research on crust layers (generally clay) situated on top of liquefiable soil, and their impact on the performance of shallow foundation structures, with the aim of quantifying the amount of ground improvement that might be needed to sufficiently reduce the effects of liquefaction on structures with shallow foundations, as this ground improvement can effectively create an artificial non-liquefiable crust layer with improved soil characteristics. Liu & Dobry (1997) identified that a thicker crust layer would reduce building settlement, because, as previously discussed, the majority of building settlement occurs during the earthquake, due to the failure of the underlying soils and dynamic soil structure interaction. A crust layer of non-liquefied soil (i.e. that hasn't failed) reduces the influence of the liquefied soil beneath on the building above. A number of studies (Karamitros et. al. 2011, Cascone & Bouckovalas 1998, Bouckovalas et. al. 2005 and Andrianopoulos et. al. 2005) have found there is a critical clay crust thickness beyond which the effect of liquefaction below this crust layer on the building is effectively zero, or at least minimal enough to accept. This critical depth is a function of the excess pore water pressures developed under the building, and is therefore dependent on the bearing pressure of the building. This allows the development of design charts to determine the depth of ground improvement required to allow the use of shallow foundations on liquefiable soils. Figure 2.9 is a design chart developed by Karamitros et. al. (2011) which allows the determination of the required clay crust thickness (H) based on the shaking intensity and the soil properties. The results of these studies are only applicable to cohesive (clay) crusts, which prevent water drainage to the surface, but can also provide conservative estimates for other crust types with greater drainage which reduces the capacity for excess pore water pressure development and therefore reduces the likelihood of liquefaction triggering.

These studies also focussed on low-rise engineered structures, assumed to be (generally) concrete or steel structures of 2-5 storeys. This kind of building is much heavier than a typical residential house in New Zealand, designed to NZS3604, which is either one or two storeys and of light timber frame construction. Because of this, houses are expected to behave quite differently from the buildings in these studies, with global settlements much closer to those experienced in the free field, as the bearing pressures caused by their foundations are not expected to be sufficient to cause largely adverse stress conditions in the ground that would lead to large settlements.

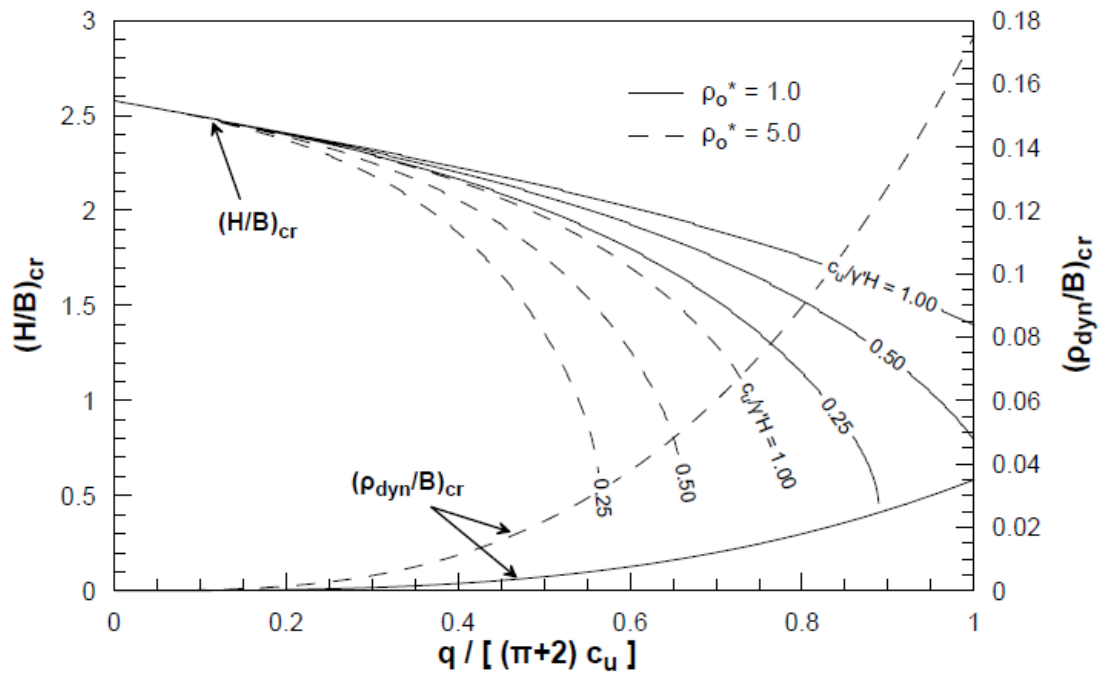


Figure 2.9. Design chart for Strip footings, with normalised critical depth $(H/B)_{cr}$ and expected settlement $(p_{dyn}/B)_{cr}$ (Karamitros et. al. (2011)).

2.4.4.2 Methods for Residential Structures

The studies discussed above offer some interesting insight into the factors affecting the settlement of low-rise structures with shallow foundations. However, apart from the design charts offered by Karamitros et. al. (2011) they do not provide any design methods, as they are based on limited numbers of centrifuge tests, or field observations. Moreover, they are not necessarily applicable to residential houses in New Zealand. This section explores methods that could be used for evaluating the performance of residential houses founded on liquefiable soil.

While large structures often require specific modelling, either of the structure itself, the foundations, the soil supporting the structure, or some combination of these, residential housing design is a far more prescriptive process, where standard requirements for bracing, connections and construction are used. This leads to a preference for simple techniques to include the interaction of building and soil to estimate the performance of residential houses on liquefied soil.

Simple punching analyses can be used to evaluate the bearing capacity during liquefaction. Several studies have been conducted in an effort to provide estimates of the residual strength of soils following liquefaction (Seed and Harder (1990), Wride et. al. (1999), Olson and Stark (2002) and Idriss and Boulanger (2007)). They provide charts of residual strength (S_v/S_r) against a corrected SPT(N) blowcount ($(N_1)_{60cs}$) (Figure 2.10). However these studies were based on the back analysis of a number of slope failures, which are not necessarily applicable to level ground conditions. They also contain a wide scatter in the data, and vary in the residual strengths for the same case histories, due to different selections of post-failure penetration resistance, other assumptions and uncertainties in general.

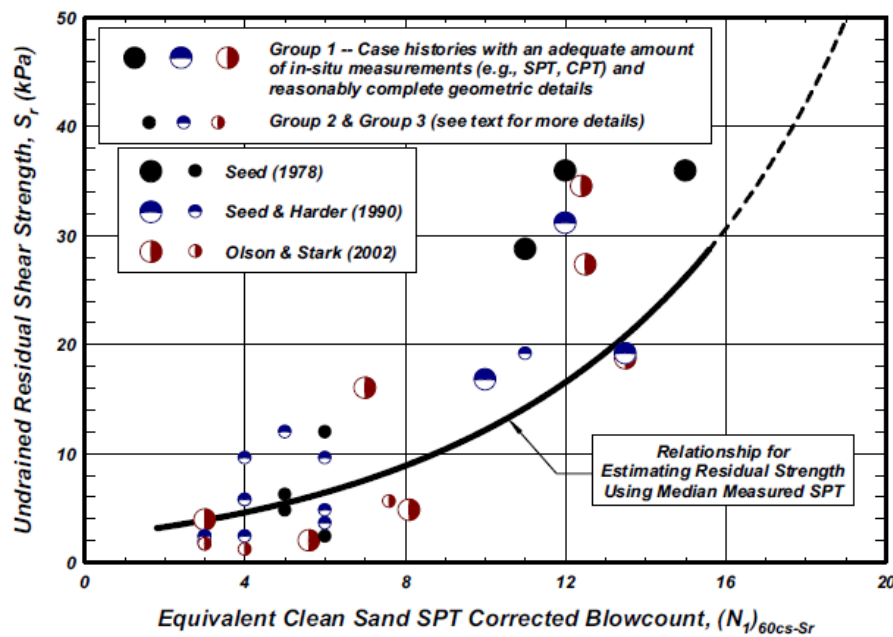


Figure 2.10. Residual shear strength, S_r , of liquefied sand vs. equivalent clean-sand, corrected SPT blow count (Idriss and Boulanger, 2007).

Ishihara (1985) proposed correlations for the surface manifestation of liquefaction based on the thicknesses of the liquefied layer, the crust thickness, and the peak ground acceleration of the earthquake. However, these are only applicable to level ground in the free field and do not provide for an estimate of damage.

It is much more difficult to estimate settlements of buildings founded on liquefiable soils. And there is very little work focussing on light buildings such as residential houses. Naesgaard et. al. (1998) gives correlations between the post-liquefaction factor of safety against bearing capacity failure, and settlement. These were produced by conducting static and dynamic computer analyses with FLAC for continuous strip footings founded on flat ground in a cohesive crust, underlain by a continuous liquefied layer. Liquefaction was induced in the analyses by setting the stress state to that of a heavy fluid ($\sigma_x = \sigma_y$, $\tau = 0$) with a much smaller shear modulus. This allowed settlements to be modelled (Figure 2.11).

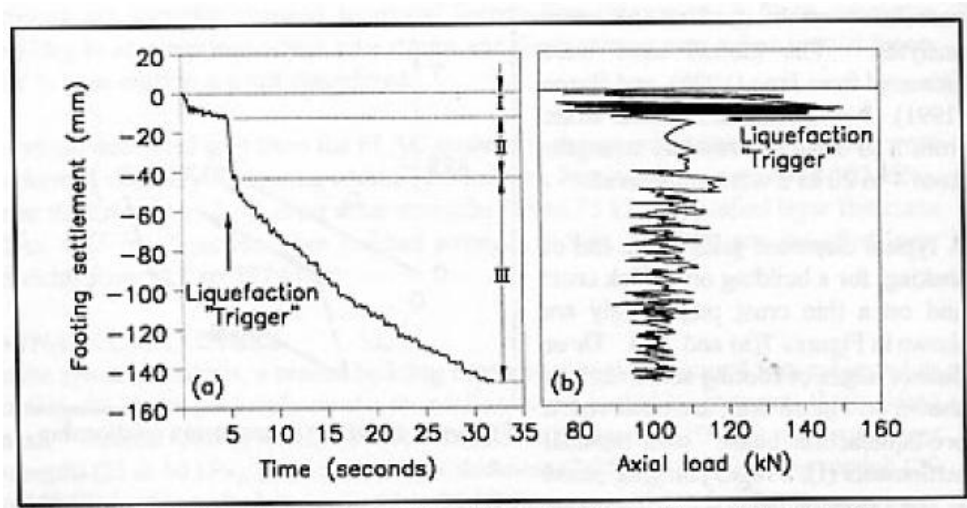


Figure 2.11. Typical footing settlement with time (a) and settlement with load (b) response for dynamic analyses (Naesgaard et. al. 1998).

Good correlations were achieved between footing settlements and the post-liquefaction factor of safety against bearing failure (Figure 2.12.(a));

$$F_s = [(2Z_c \times c_u) + (5.14\tau_{RES} \times B)]/Q_l$$

where:

- Z_c = crust thickness
- c_u = crust shear strength
- τ_{RES} = liquefied layer residual strength
- B = footing width
- Q_l = footing load per unit length

This correlation was improved with trial and error (Figure 2.12.(b)) by including the limiting shear strain (γ_{lim}), the thickness of the liquefied layer (Z_l) to give

$$X_s = F_s / (\sqrt{Z_l} \times \sqrt{\gamma_{lim}} \times \sqrt[4]{Q_l})$$

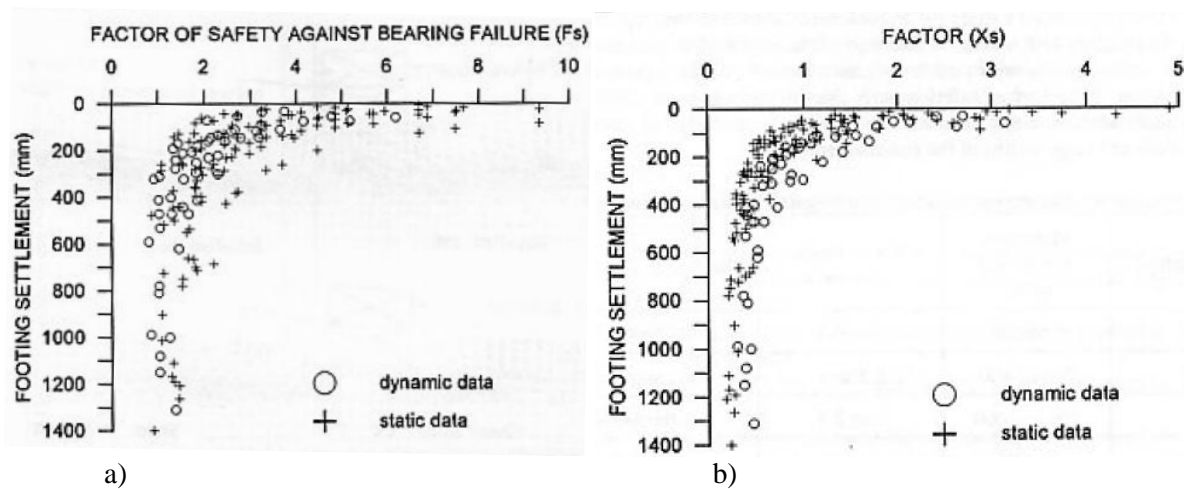


Figure 2.12. a) Post liquefaction factor of safety, F_s and footing settlement. b) Post liquefaction factor X_s and footing settlement (Naesgaard et. al., 1998).

The settlements here are only due to the weight of the building bearing on the soil, and do not include post-liquefaction consolidation due to the dissipation of excess pore water pressure (i.e. those dealt with in Section 2.4.3), or any additional settlements due to ejected sand and water. Naesgaard notes that “further work is required to assess the influence of ground slope, topographical features, foundation shape, variations in shaking, time of liquefaction occurrence, intermittent liquefaction, cohesionless crust and pore pressure migration”.

To the knowledge of the author, there is no research available that attempts to combine the three different types of settlements mentioned above; that from the building weight, the dissipation of excess pore pressure, and the ejection of sand and water during liquefaction in a simple design procedure. There is also no research on the level of differential settlements that may result from non-uniform building weights or changing soil profiles, nor anything that will give forces induced in the foundation as a result of these ground movements.

2.5 Current State of Practice

The current timber-framed buildings standard, NZS3604:2011 (Standards New Zealand, 2011), and all previous versions of this standard, contain no specific requirements for residential buildings founded over liquefiable soil, either in the design of foundations to account for this, or treatments to soil to reduce the effects of liquefaction. The only provision is that foundations are founded on ‘good ground’ with an ultimate bearing capacity of 300kPa, where there is no evidence of land slips or ground creep, and no buried organic topsoil, soft peat, or soft or expansive clay. But this is only for static ground conditions, and does not deal with the possibility of liquefaction. This is despite the likelihood of liquefaction occurring in a major seismic event in Christchurch being well known before the Canterbury Earthquakes.

2.5.1 Review of New Zealand Design Standards

This section reviews the significant changes in New Zealand housing standards from before the first standards were introduced to the present day. The four standards N.Z.S.S.95:1935, NZS1900:1964, NZS3604:1981 and NZS3604:1999 were chosen as they represent the general standards of foundation construction in each of the five time periods used by BRANZ in their studies: pre-1930, 1930-1959, 1960-1979, 1980-1999 and post-2000 (Beattie et. al., 2011; Liu & Beattie, 2012; Thomas & Shelton, 2012 and Thomas et. al., 2013). Although there were other standards brought out during these periods, and amendments to the specified standards during these time periods were made, the changes made were minimal, and the chosen standards offer a good representation of foundation construction in each of the time periods. For summary drawings showing the specific requirements of the standards corresponding to each time period, see Appendix A.

2.5.1.1 Pre-1930

Before the 1930s there were no residential housing building standards in New Zealand. All houses inspected for this research in the pre-1930s age bracket were concrete perimeter with short piers. The concrete perimeters were constructed of generally low-grade concrete, with uniformly-graded aggregate, and often used fill such as bricks, bits of old concrete or large stones up to 200-300mm diameter. These perimeter walls were unreinforced. The pier supports were either concrete or timber. If concrete they could be precast or cast in-situ. They are never rigidly fixed to the sub-floor structure, and in some cases are not fixed at all.

2.5.1.2 1930-1959 (N.Z.S.S.95:1935)

Housing standards in New Zealand began in the 1930s with the introduction of N.Z.S.S. 95 in 1935 following the 1931 Napier earthquake. This was revised in 1939 and 1955. This early standard introduced a minimum founding depth below the ground of 1' (305mm) for perimeter walls. It also set minimum height requirements for concrete perimeter foundations based on the height of the wall being supported (Table 2.2). Reinforcement requirements were also introduced. These had a consistent layout in terms of number of bars, one inside each corner of the concrete perimeter section, with ¼" (6mm) stirrups placed at 24" (600mm). The size of bars to be used also changed with the height of the wall being supported, as shown in Table 2.2. These reinforcement requirements were quite significant when compared to today's standards, possibly in reaction to the devastating damage caused by the Napier earthquake and are shown in Figure 2.13. For full foundation details see Appendix A or N.Z.S.S.95:1935.

Table 2.2. Perimeter wall characteristics of N.Z.S.S.95:1935.

Wall Height	Longitudinal reinforcing diameter	Minimum Vertical Thickness
≤ 15' (4.57m)	½" (12mm)	9" (230mm)
≤ 25' (7.62m)	⅝" (16mm)	12" (305mm)
≤ 40' (12.19m)	¾" (19mm)	15" (380mm)

There were no specific requirements laid down for residential slab-on-grade foundations in the N.Z.S.S.95 standards and there was only one slab-on-grade house inspected during this research that fell into the 1930-59 category.

There was also no mention of a required concrete strength or a minimum bearing capacity upon which the foundation was constructed.

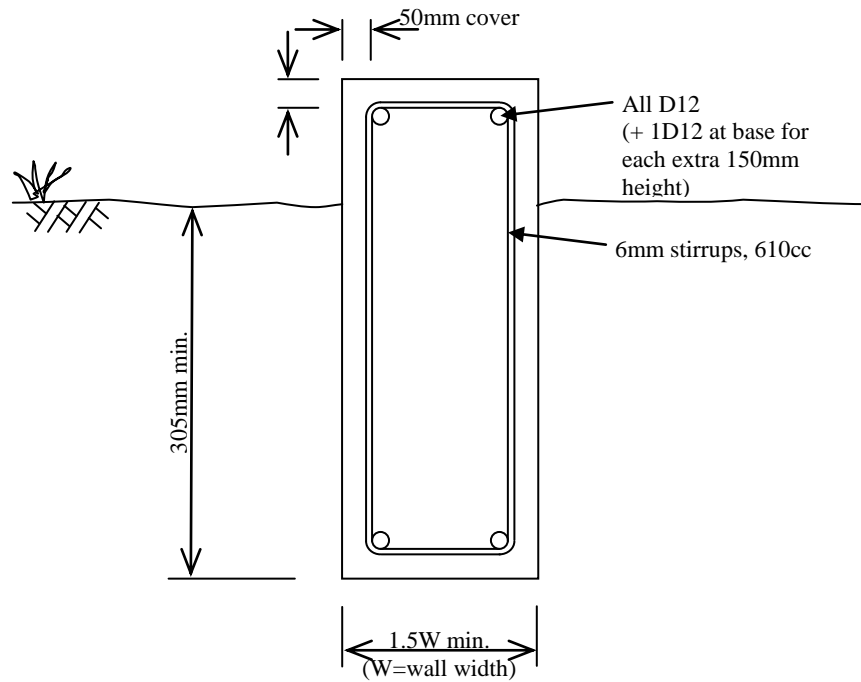


Figure 2.13. Concrete perimeter section requirements for one-storey house from N.Z.S.S.95:1935.

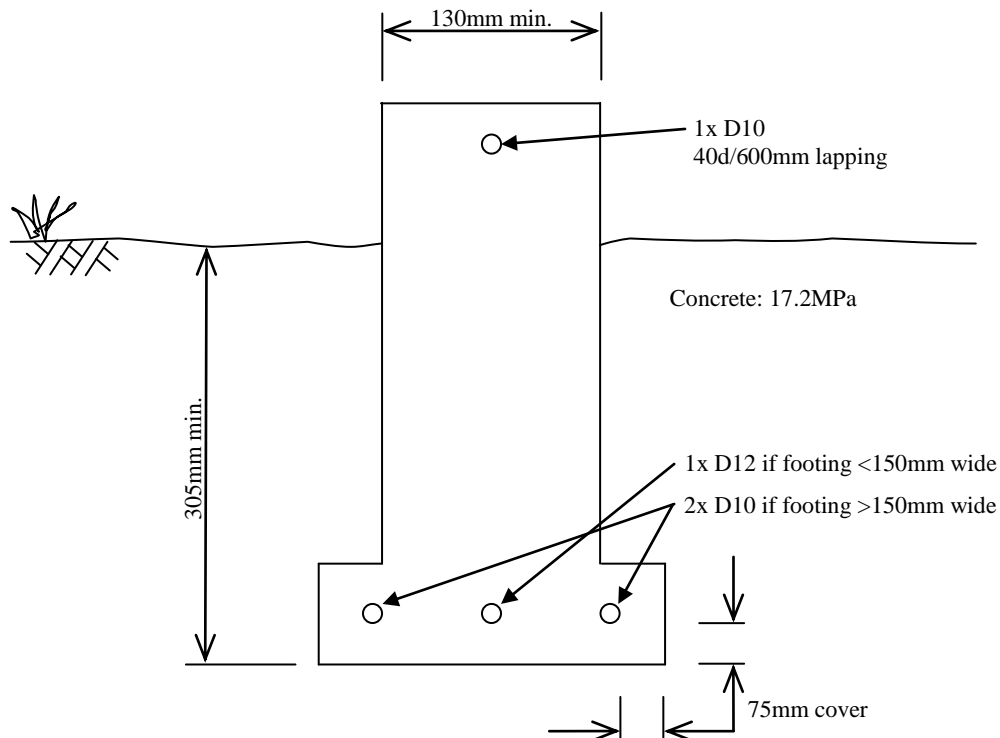
2.5.1.3 1960-1979 (NZS1900:1964)

N.Z.S.S.95 was replaced in 1964 by the first NZS1900. It was the first standard to set a value for minimum concrete strength of 2500psi (17.2MPa) and for soil bearing capacity, stating that if the ground could not support more than 1ton/sq.ft. (107kPa) then foundation walls were required. It also stated that perimeter footings must have adequate bearing area to safely support the imposed loads from the house, taking into account the character of the soil. It brought in minimum and maximum height requirements of 6" (150mm) and 4'6" (1.4m) respectively for the under-floor piers. With maximum spacing of 4'6" (1.4m). NZS1900:1964 was the first to separate foundation construction requirements between brick construction and timber-framed buildings. Lightweight structures could use footings made from concrete or brick with different requirements based on building height and intended width of footing as shown in Table 2.3. Brick walls were required to be supported by a reinforced concrete foundation wall with more stringent dimensional and reinforcement requirements (Table 2.3). There was a requirement for all building elements to be tied and interconnected and foundations were required to be tied to members which could resist horizontal loads in both directions, both tension and compression, equal to 10% of the maximum vertical load. An example concrete section is shown in Figure 2.14. For full sections requirements see Appendix A or NZS1900:1964.

NZS1900:1964 still had no specific requirements for residential slab-on-grade foundations and there were only three houses in this age category inspected during this research.

Table 2.3. Perimeter footing characteristics of NZS1900:1964.

Building Type	Brick Footing	Reinforced Concrete Footing	
		Reinforcement Requirements	Additional
1-storey	>8" (200mm)	W<6" (150mm): 1x ½" (12mm) rod	If W>5", 1x ⅜" (10mm) rod in top
		W>6" (150mm): 2x ⅜" (10mm) rods	
2-storey		W<8" (200mm): 2x ⅜" (10mm) rods	If W>6", 1x ½" (12mm) rod in top
		W>8" (200mm): 3x ⅜" (10mm) rods	
Brick cladding	N/A	(at least) 2x ½" (12mm) rods in centreline, one top and bottom	⅜" (10mm) ties at 24" (600mm)

**Figure 2.14. Concrete perimeter section requirements for one-storey house from NZS1900:1964.**

2.5.1.4 1980-1999 (NZS3604:1981)

NZS1900 was replaced by NZS3604:1981. This standard gave separate minimum depths of embedment for perimeter footings of 450mm for expansive clay and 300mm for other soils. The minimum required bearing capacity remained at 100kPa. It also gave different minimum and maximum heights above ground for timber and concrete piers, and prescribed required footing dimensions for these to be cast-in-place into the ground. There was now also a defined requirement for a certain level of attachment of piers to the subfloor structure. And minimum concrete strength for all concrete foundation components was changed slightly to 17.5MPa at 28 days.

For the perimeter footings, longitudinal reinforcement requirements remained similar, and a maximum height of 2m was introduced. Minimum widths were also given directly in millimetres for the first time, where previously they had been expressed as multiples of the supported wall width, W. This was the first time the deformed bars had been specified for reinforcement as opposed to just

‘rods’ which were often round bars. There was now the option to use either deformed bars or steel wire mesh as transverse (and additional central longitudinal) reinforcing. An example section is shown in Figure 2.15.

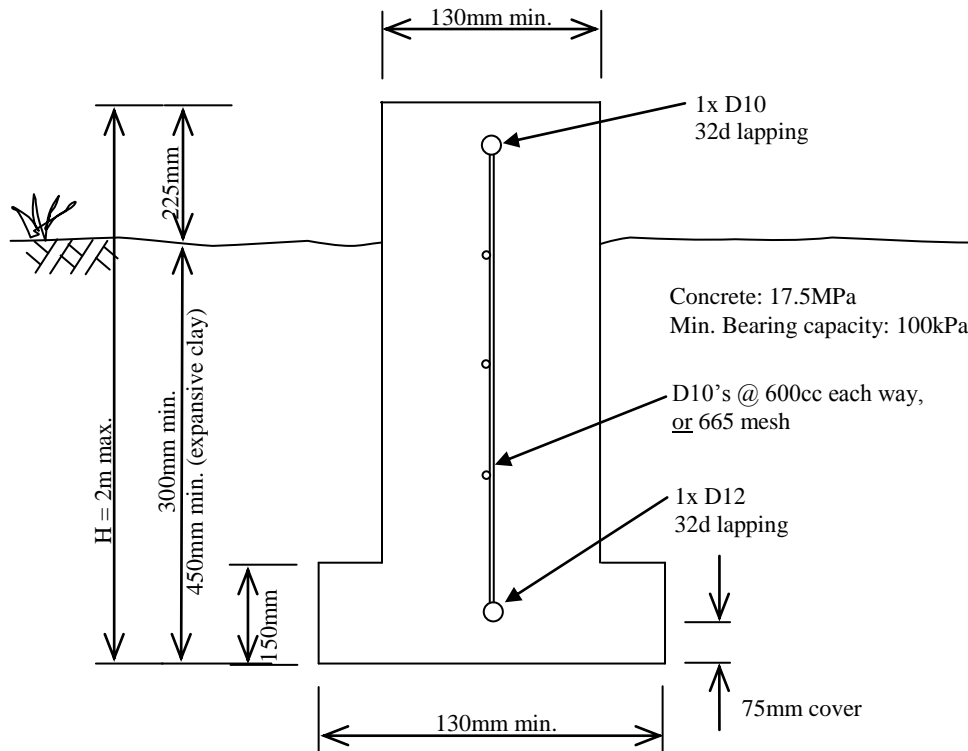


Figure 2.15. Concrete perimeter section requirements for one-storey house from NZS3604:1981.

NZS3604:1981 was the first standard to give specific requirements for residential slab-on-grade foundations. Minimum slab heights above ground were given, as well as the requirement for a continuous perimeter foundation wall of minimum depth, and a continuous vapour barrier underneath. An underlying granular base was also specified with set thicknesses and allowable particle size distributions. There was no specific requirement for the level of compaction of the fill, except that it had to be placed on a “solid bottom or firm fill” in 100mm thick layers. Minimum slab thicknesses and steel wire mesh reinforcing contents were also given. Thickening of the slab under load-bearing walls was also required. Figure 2.16 shows an example section, for full section requirements see Appendix A or NZS3604:1981.

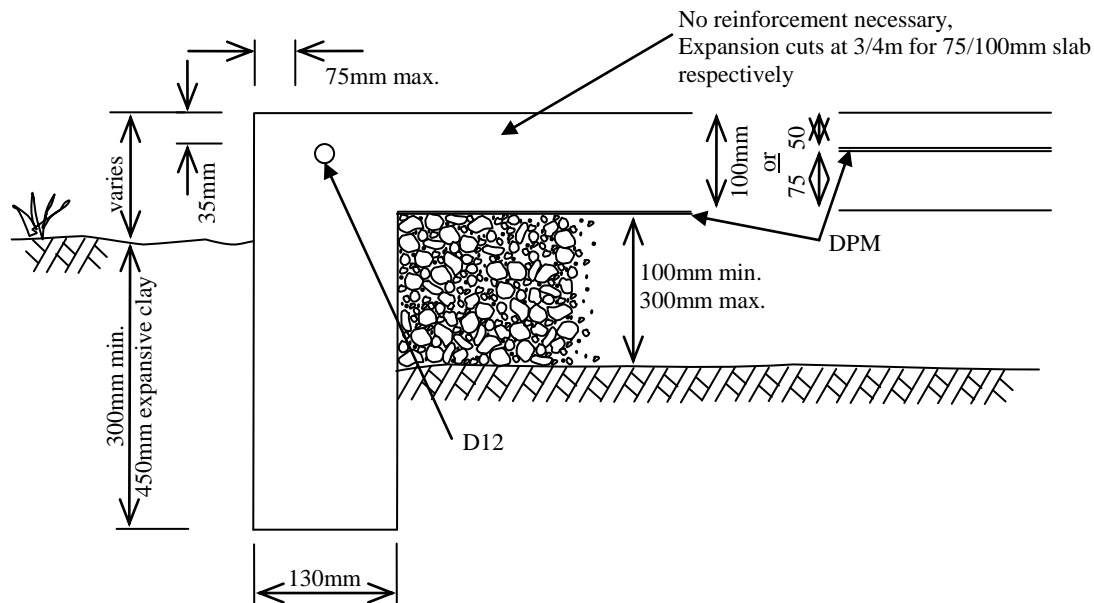


Figure 2.16. Concrete slab-on-grade requirements for one-storey house from NZS3604:1981.

2.5.1.5 Post-2000 (NZS3604:1999)

The 1999 version of NZS3604 was the first one to increase the allowable bearing capacity of founding soils to 300kPa. If this requirement is not satisfied, specific engineering design is required for the foundations. However it allowed ‘good ground’ of 300kPa to be assumed if there is no evidence of buried services, slips, earth fill, organic topsoil, soft peat or clay, or if the site is adjacent to similar buildings which are deemed to be okay (i.e. have not suffered any ill effects due to poor ground conditions).

For perimeter foundations (Figure 2.17), the minimum depth of embedment was reduced from 300mm to 200mm. Some new height restrictions were brought in for various types of piles but apart from that everything else remained basically the same.

For slab-on-grade foundations (Figure 2.18) the requirements also remained basically unchanged. The allowable dimensions of the granular base were relaxed somewhat. It could now be laid down in 150mm layers, but a requirement was introduced that each layer had to be compacted “until the material is tightly bound together and does not visibly deform under the weight of a pressed adult heel”. New maximum dimension limits were also introduced for shrinkage control of the different types of concrete (reinforced, un-reinforced and fibre-reinforced).

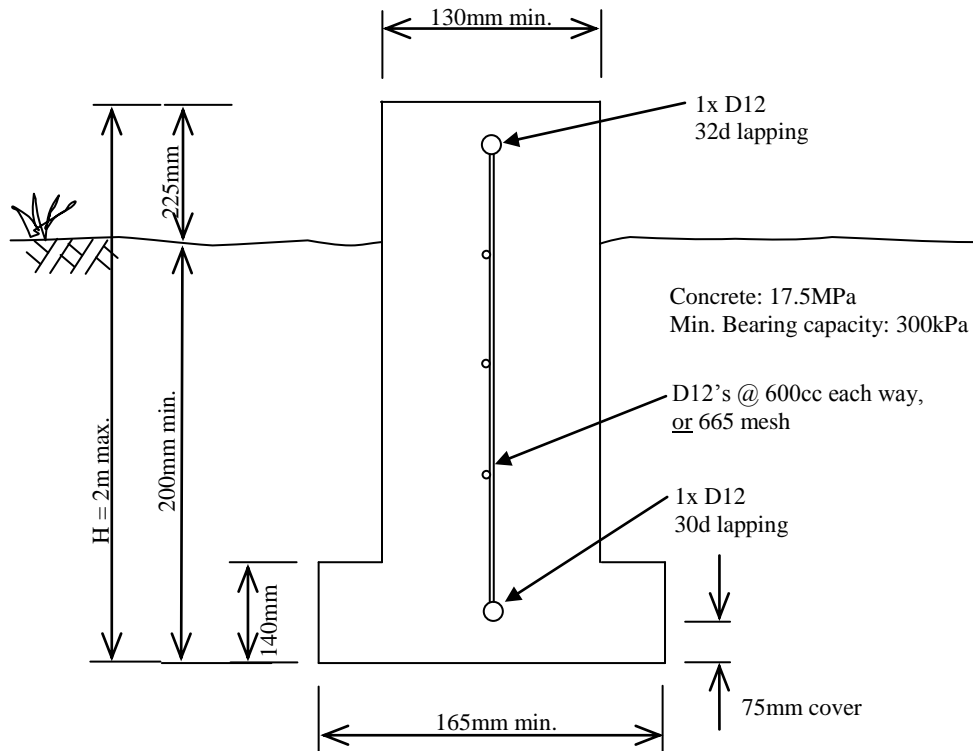


Figure 2.17. Concrete perimeter section requirements for one-storey house from NZS3604:1999.

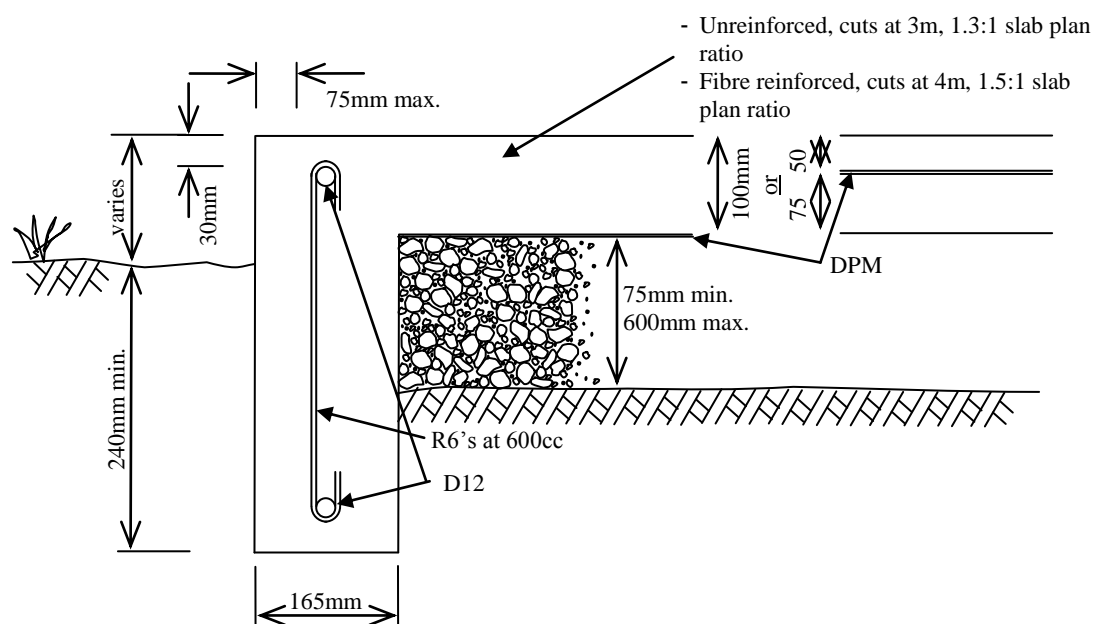


Figure 2.18. Concrete slab-on-grade requirements for one-storey house from NZS3604:1999.

2.5.1.6 RibRaft™ Specifications

RibRaft™ is a proprietary flooring system designed and produced by Firth Industries and is classed as a special engineering solution for residential floors. It provides higher strength and stiffness than a standard NZS3604 slab-on-grade and better thermal insulation. Firth 2003 outlines the specifications for RibRaft™ floors with an 85mm mesh-reinforced concrete slab overlying a grid 1100mm square

polystyrene foam pods, separated by a grid of reinforced concrete ribs 100mm wide internally and 300mm wide around the perimeter and under load-bearing walls. Figure 2.19 shows the specifications for RibRaft™ slabs.

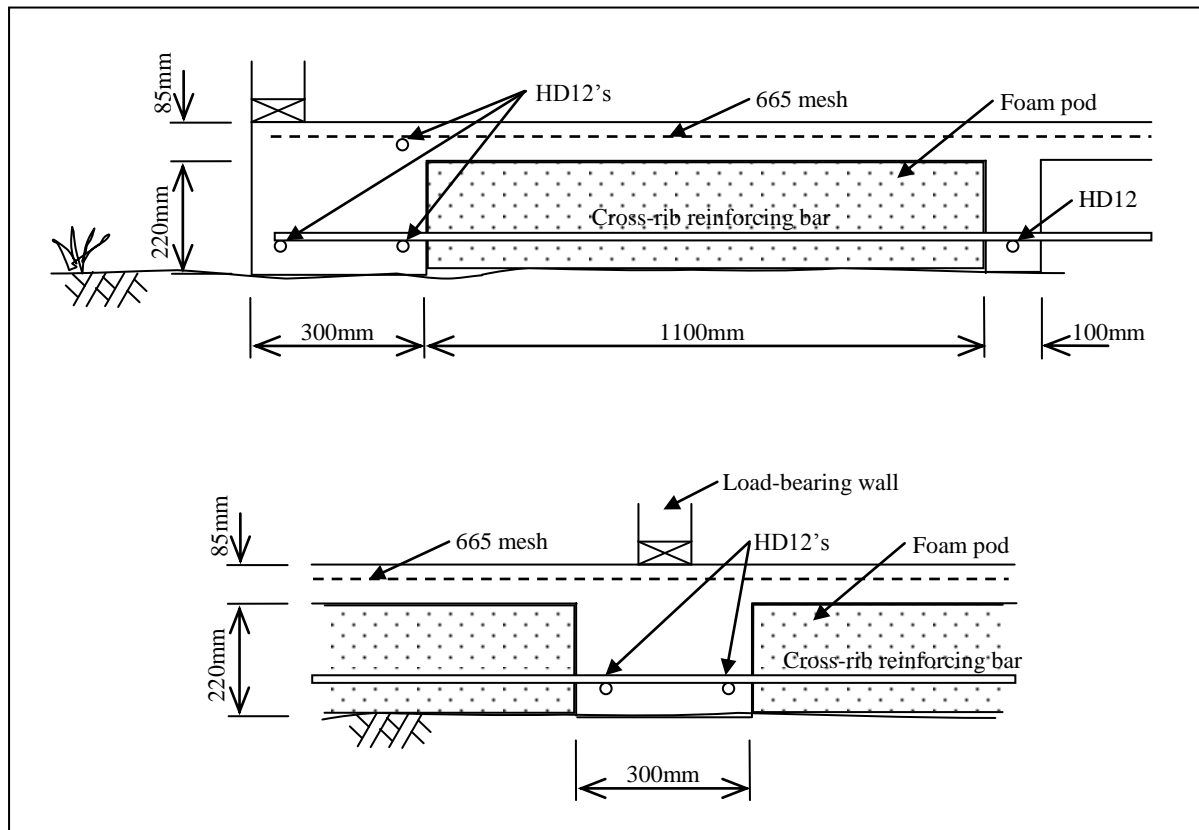


Figure 2.19. RibRaft™ technical specifications. Exterior and interior ribs (top) and load-bearing wall details (bottom). Firth 2003.

2.5.1.7 Floor Level Tolerances for New Construction

There are a number of standards which outline different measures for the allowable variation in floor level of newly-constructed foundations. For serviceability conditions, Appendix B of Verification Method B1/VM4 (NZS3604:2011) sets an allowable probable maximum differential settlement of 25mm over 6m, corresponding to a 1/240 slope, or 0.4%.

Table 2.1 of NZS3604:2011 states that for timber framing, the maximum deviation from the horizontal is 5mm in 10m, or a total of 10mm over any length greater than 10m. In Table 2 of the old standard NZS3124:1987, the variation in bearing surfaces for timber must be within $\pm 5\text{mm}$, with a maximum depression between two high spots 3m apart of 8mm and a maximum floor slope of 0.53% or 1/190. However, this standard is no longer current or referred to by NZS3604.

As outlined in the various guidance documents from DBH and MBIE, the Engineering Advisory Group conducted a survey in early 2011 of new concrete slab foundations. They found an overall variation in floor level in new slabs between 15-20mm, with average slopes between points 2m apart

of 0.35-0.65% or 1/300-1/150. This survey helped to produce Figure 2.20 of the indicator criteria for different foundation types needing a foundation repair or rebuild.

Column 1	Column 2	Column 3	Column 4	Column 5
Floor type	NO foundation relevel considered necessary	Foundation relevel indicated	Foundation rebuild indicated (Partial or full)	House rebuild may be indicated
Type A Timber-framed suspended timber floor structures supported only on piles	The slope of the floor between any two points >2 m apart is <0.5% (1 in 200) [Note a] and The variation in level over the floor plan is <50 mm	The variation in floor level is >50 mm and <100 mm Note that the floor relevel is expected to be achieved by packing the piles	The variation in floor level is >100 mm [Note c] over the floor plan or The floor has stretched >50 mm [Note d] Note that full or partial re-piling is expected to be undertaken to achieve a level floor	The house has fully or partially collapsed off the piles and repair may be uneconomic This will relate to the degree of superstructure damage [Note f]
Type B Timber-framed suspended timber floor structures with perimeter concrete foundation	The slope of the floor between any two points >2 m apart is <0.5% (1 in 200) [Note a] and The variation in level over the floor plan is <50 mm	The variation in floor level is >50 mm and <100 mm [Note b]	The variation in floor level is >100 mm [Note c] over the floor plan or The floor has stretched >20 mm [Note e]	The house has fully or partially collapsed off the piles and repair may be uneconomic This will relate to the degree of superstructure damage [Note f]
Type C Timber-framed dwelling on concrete floor	The slope of the floor between any two points >2 m apart is <0.5% (1 in 200) [Note a] and The variation in level over the floor plan is <50 mm and There are no cracks in ceramic floor tiles and There is no distress in vinyl floor coverings or carpet	The variation in floor level is >50 mm and <150 mm and Services are functioning	The variation in level over the floor plan is >150 mm or There is irreparable damage to buried services within the house footprint	This will relate to the degree of superstructure damage [Note f]

Note: Criteria are indicators only – a full assessment of the damage is required to make decisions on whether to repair or rebuild.

Figure 2.20. Indicator criteria for floor/foundation re-level or rebuild (reproduced from Table 2.3, MBIE 2012).

2.5.2 Guidance From Outside of New Zealand

Around the world there is some guidance on how to go about the design of residential buildings on liquefiable ground. Naesgaard et. al. (1998) states that as a practising engineer in Vancouver, Canada, when designing light structures on a crust over liquefiable soils, it is customary to design foundation loads to be less than the punching shear capacity of the crust. Settlements are estimated using volumetric strain correlations from Tokimatsu and Seed (1987) or Ishihara and Yoshimine (1992) with some form of increase for shear strain. The correlations for surface manifestation by Ishihara (1985) are also sometimes used.

A 2007 task force (Anderson et. al., 2007) produced geotechnical design guidelines for buildings on liquefiable sites for the Greater Vancouver region. This includes recommendations on the choice of seismic hazard, earthquake magnitude and ground motion records for design. It contains guidelines for the determination of liquefaction resistance and post-liquefaction response of soils, and tolerable displacements for design. It also contains suggestions for ground improvement measures and structural modifications to reduce the effects of liquefaction. It recommends the use of the procedures discussed above in Section 2.4 for the calculation of settlements and lateral spreading in the free-field due to liquefaction. For light buildings founded on crusts over liquefiable soil, it recommends the relationships from Naesgaard et. al. (1998). For differential settlements it suggests the use of half the total settlement for a relatively uniform soil profile across the site from Martin et. al. (1999), a similar document developed for California.

These two documents are a good start to regulating the inclusion of liquefaction hazards in engineering design. However, neither of them is specific to residential buildings, and this is a large area where improvements can be made.

The Japanese Geotechnical Society published the Case History manual of post-liquefaction remediation measures (Technical Committee, 2001). This documents the methods used to repair damage to houses following liquefaction during the 1993 Hokkaido-nansei-oki and 1995 Hyogoken-nambu earthquakes. Following the 4th September 2010 and 22nd February 2011 earthquakes, the Ministry of Business, Innovation and Employment, MBIE (formerly the Department of Building and Housing) published detailed, house-specific documents with guidance and recommendations on how to repair damaged houses using established methods. These various documents show that repair methods are well understood, and able to be adapted from current practice for house relocations and the like. However there is limited knowledge on how to prevent or reduce damage in the first place.

2.5.3 Guidance on House Construction Following the Canterbury Earthquakes

Following the release of the first two documents by the Department of Building and Housing on the repair and rebuild of houses affected by the Canterbury Earthquakes, extensive field investigations and design were carried out to improve this advice. This resulted in the latest document from the MBIE, “Repairing and Rebuilding Houses Affected by the Canterbury Earthquakes”, which was released in December 2012 and is continually updated. This document expands on the content in the previous two documents on the repair and re-build of damaged houses, but also provides new requirements for the construction of new houses in Christchurch.

The government has split the residential areas of Christchurch located on the flat into three land zones, or ‘Technical Categories’, not including the residential red zone. These are shown in Figure 2.21 and are based on extensive geotechnical testing of the land damage caused by liquefaction during the Canterbury earthquakes.

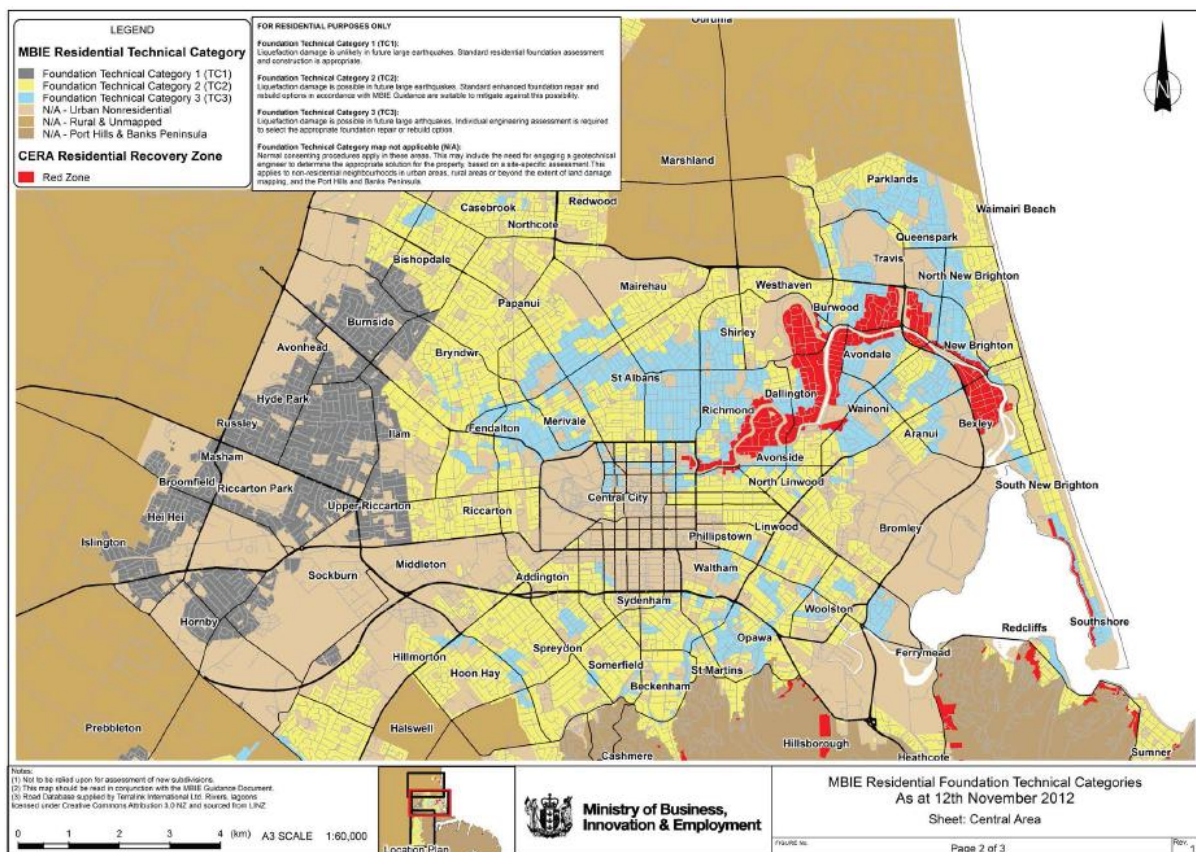


Figure 2.21. Residential zoning in Christchurch (reproduced from MBIE 2012).

The three categories are characterised as follows:

- TC1: Land damage from liquefaction unlikely in future large earthquakes. Standard residential foundation assessment and construction to NZS3604 is appropriate. Should have had no surface damage, liquefaction or land settlement.
- TC2: Liquefaction damage is possible in future large earthquakes. Shallow ground investigations may be required. May have had cracks less than 50mm wide, small amounts of liquefaction, or land settlement less than 100mm.
- TC3: Liquefaction damage is likely in future large earthquakes. Geotechnical engineering assessment may be required to select the appropriate foundation repair or rebuild. Suffered greater land damage than TC1 or TC2 and requires specific engineering assessment.

Under these guidelines, geotechnical investigation and specific engineering design and assessment are required for repairs, re-building or new construction of residential buildings located within TC3 areas. Guidelines are also provided for the levels beyond which repair or rebuild of different foundation types is required:

- Piles: 15mm in 1m height tilt = replace pile
- Timber floors (with or without concrete perimeter):
 - o Floor re-level: >10mm tilt over 2m, or 50-100mm tilt over floor width
 - o Floor re-build: >20mm tilt over 2m (concrete perimeter), or >100mm tilt over floor width
- Concrete slab:
 - o Floor re-level: >50mm tilt over floor width & services ok
 - o Floor re-build: >150mm over floor width &/or services not ok
 - o Cracks <1mm wide: no structural repair, seal to weather
 - o Cracks 1-10mm wide: fill with epoxy
 - o Cracks 10-20mm wide: fill with grout
 - o Cracks >20mm wide: break out slab along crack and re-cast section

The guidance recommends the use of light or medium weight claddings for walls and roofs, with heavyweight claddings only permissible on some foundation systems. Design guidelines are also provided as to suitable foundation options for each Technical Category.

This document is a good attempt at providing engineering guidance on the repair, rebuild and construction of new houses for the wide area of a whole city such as Christchurch. It provides simple and effective foundation solutions for a range of land conditions which allow faster and cheaper construction, recognising that not all areas were affected by liquefaction, and that in these areas the current housing standards provided adequate structures. Site specific investigation is required for sites

that may suffer significant liquefaction in future large seismic events, which will ensure the future robustness of the Christchurch and Canterbury housing stock.

Although this guidance has been produced specifically for Christchurch, the range of innovative and effective foundation solutions that have been provided can easily be used in other parts of New Zealand where liquefaction possibilities are recognised.

3 The Performance of Houses from Inspections after the 4th September 2010 Darfield Earthquake

3.1 Introduction

The 4th September 2010 M_w 7.1 Darfield earthquake subjected the majority of Christchurch to peak ground accelerations between 0.15-0.2g (O'Rourke et. al., 2012), and caused widespread liquefaction in Christchurch and some surrounding towns. This liquefaction caused significant damage to a number of houses in the affected suburbs. Following this earthquake, a joint summer research programme between the University of Canterbury and the University of Auckland was organised.

Under this programme, around 500 houses were inspected by different teams in three of the suburbs hit worst by liquefaction effects: Avonside, Bexley and North Kaiapoi. The inspections were all conducted prior to the 22nd February 2011 Christchurch earthquake. There was a high density of inspections conducted within these three suburbs to understand the performance of and damage sustained by houses subjected to high levels of liquefaction and lateral spreading.

The inspections recorded the levels of damage to different parts of the buildings, including the foundations, structure and internal and external partitions. Measurements of crack widths and building tilts were taken where possible to quantify the damage. Damage to the surrounding property was also recorded. All areas of damage were photographed for further reference.

This chapter presents the typical damage that occurred in residential buildings affected by liquefaction, and analyzes the results of the inspections with regard to the overall performance of houses subjected to severe liquefaction effects.

3.2 Locations Surveyed

The research project focussed on the areas that were the worst-hit in the 4th September 2010 earthquake; the suburbs of Avonside, Bexley and North Kaiapoi (Figure 3.1). Table 3.1 shows the total number of houses surveyed in each of the three suburbs, and which foundation types were inspected.

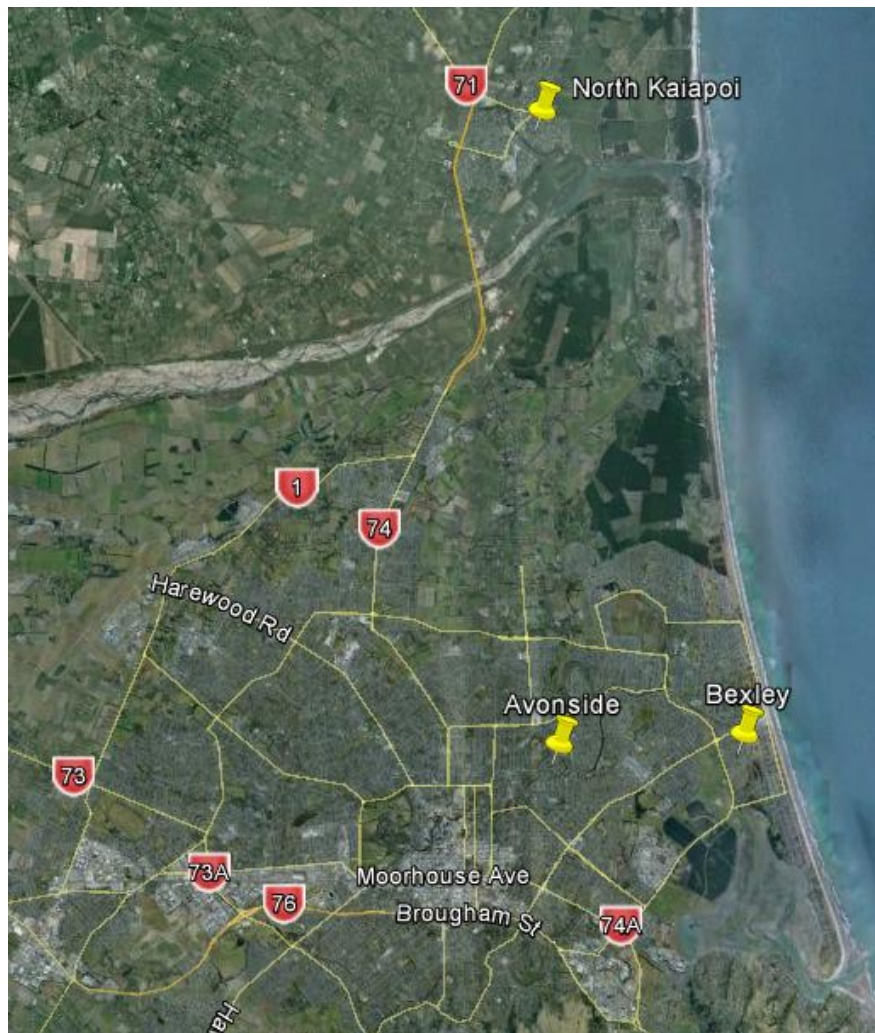
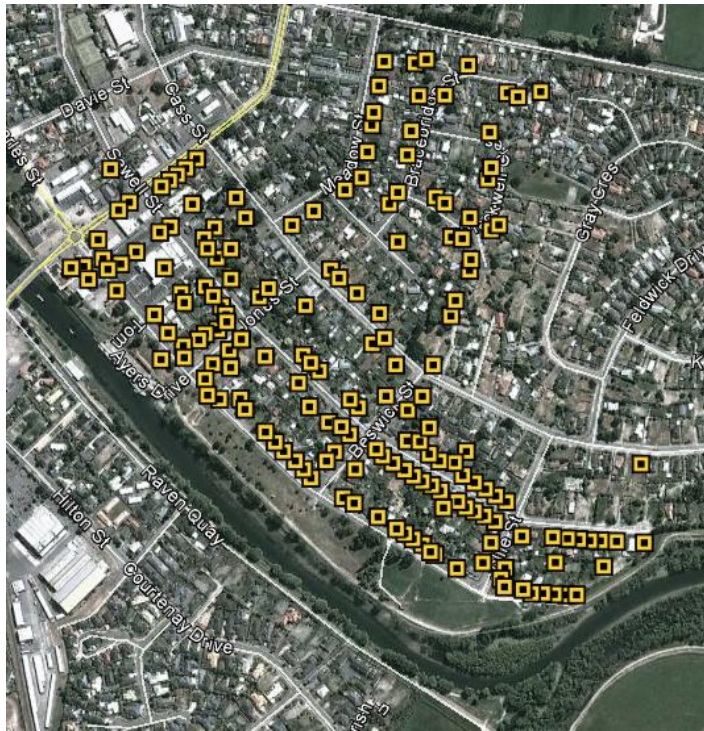


Figure 3.1. Locations of suburbs badly affected by liquefaction where houses were inspected following the 4th September 2010 earthquake.

Table 3.1. Number of houses surveyed in each suburb and foundation type.

Suburb	Number of Buildings Surveyed			
	Concrete Perimeter	Slab-on-grade	Unknown/Other	Total
Avonside	99	8	3	110
Bexley	71	124	5	200
Kaipoi	106	61	15	182
Total	276	193	23	492

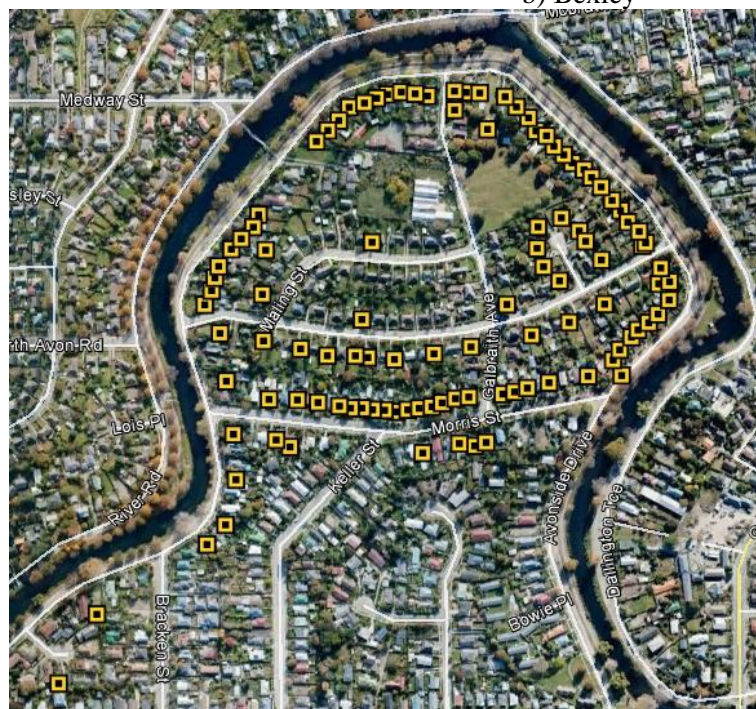
Figure 3.2 shows the locations of the individual houses inspected in each suburb. Within each suburb, a sub-area was chosen where almost every building was inspected. This provided an intense concentration of houses that all experienced very similar shaking characteristics in the 4th September 2010 earthquake (within a suburb), allowing direct comparison between the level of damage seen with the level of liquefaction or lateral spreading experienced.



a) Kaiapoi



b) Bexley



c) Avonside

Figure 3.2. Individual inspection locations for each suburb.

3.3 Methodology

3.3.1 Survey Equipment

Table 3.2 shows the equipment that was used in site visits following the 4th September 2010 earthquake.

Table 3.2. Field survey equipment.

Equipment	Details
Safety Gear	Hard hat, Steel-cap boots, High-visibility vest
Camera	For photographing damage
Survey Form	See Section 3.3.2.1 and Appendices C and D
Clipboard	To allow accurate recording of data during the survey
Tape-measure and Ruler	To measure settlement and crack widths
Digital Inclinometer	For measuring floor and tilt angles (Figure 3.3)



Figure 3.3. Digital inclinometer.

3.3.2 Inspection Process

Each building inspection followed the same steps, outlined below:

- 1) Permission to conduct an inspection was obtained from the house owner. This was done by door knocking. Formal information letters explaining the purpose and process for the inspection were given to the residents where requested. If no one was at the house, a request letter (Appendix B) was left and a further visit scheduled for a later date.
- 2) A site walk-over was conducted to identify the main damage areas to the structure and surrounding land. These observations were recorded with sketches, notes and photographs. The photos taken included, but were not limited to:
 - An overall view of the house on all sides
 - Foundation type and any visible damage to this
 - Structural damage (e.g. Framing, walls, roofing)
 - Land damage (e.g. lateral spreading cracks, sand boils and other evidence of liquefaction)
- 3) The survey form was then filled in. This involved ticking boxes to indicate the level of damage to each element, writing descriptions of the damage, and sketching a floor plan to

indicate the location of any major damage and measurements taken. See Appendices C and D for the inspection forms used.

3.3.2.1 Data Recorded

Table 3.3 shows the information that was put into a spreadsheet for each house.

Table 3.3. Data recorded during inspections, showing possible values for the appropriate indices.

Index	Description
House ID	A unique number to allow the house to be identified without the address
Street Address	Street name and number
Date of Inspections	
Names of Inspectors	
EQC Damage Sticker	Sticker colour, if there was one
House Description	Number of storeys, foundation type, cladding, framing and roofing materials
Liquefaction Severity	None, Low, Moderate, High
Lateral Spreading Severity	None, Low, Moderate, High
Differential Settlement	None, Low, Moderate, High. Based on any slope or settlement values in mm recorded
Total Settlement	None, Low, Moderate, High
Foundation Damage	None, Low, Moderate, High
Structural Damage	None, Low, Moderate, High
External Partition Damage	None, Low, Moderate, High

3.3.2.2 Ratings Criteria

Once the inspections had been completed, the data was collated and entered into a summary spreadsheet. Table 3.4 lists the damage severity criteria that were used by the summer research team during the data collation stage to rate the level of damage under various categories for each property. This data collation was carried out by the same two researchers for all inspected houses. They input all the inspection data into spreadsheets, and used the criteria in Table 3.4 to ensure the ratings made during each inspection were consistent between houses. They drew information from the inspection forms, photos and preliminary spreadsheet data to come to the final decision. This was done because the inspections were conducted by approximately 10 different people, which may have led to inconsistencies in the data.

Table 3.4. Severity ratings for each damage index with descriptions for each.

Liquefaction Severity	
High	Extensive sand boils with thickness > 200 mm (very severe) or thickness of 50 – 200 mm (severe) and/or ground failure/significant ground distortion/large fissures/cracks with evidence of soil liquefaction.
Moderate	Isolated but relatively large in size sand boils (0.5 – 1.0 m in diameter) and/or some ground distortion (small cracks) with evidence of liquefaction.
Low	Scattered/isolated and small in size sand boils with no visible ground distortion.
None	No liquefaction
Lateral Spreading Severity	
High	$Crack\ width \geq 300\ mm$ (very severe), $300\ mm > crack\ width \geq 100\ mm$ (severe)
Moderate	$100\ mm > crack\ width \geq 20\ mm$
Low	$Crack\ width < 20\ mm$
None	No cracks
Differential Settlement Severity	
High	$\theta \geq 1/200$
Moderate	$1/200 > \theta \geq 1/400$
Low	$\theta < 1/400$ (for $L > 2m$)
None	Zero
Total Relative Settlement Severity	
High	Settlement $\geq 100\ mm$
Moderate	$100\ mm > settlement \geq 30\ mm$
Low	Settlement $< 30\ mm$
None	None
Foundation Damage Severity	
High	Large cracks, individual crack width $> 5\ mm$, cumulative crack width $> 30\ mm$
Moderate	Cracks, $5\ mm \geq ICW > 2\ mm$, $30\ mm \geq CCW > 5\ mm$
Low	Hair line cracking, $ICW \leq 2\ mm$, $CCW \leq 5\ mm$
None	No damage or cracks
Partition Damage Severity	
High	Large cracks, individual crack width $> 5\ mm$, cumulative crack width $> 30\ mm$
Moderate	Cracks, $5\ mm \geq ICW > 2\ mm$, $30\ mm \geq CCW > 5\ mm$
Low	Hair line cracking, $ICW \leq 2\ mm$, $CCW \leq 5\ mm$
None	No damage or cracks

The severity of structural damage did not have specific ratings criteria, and was judged during the inspections by the researchers based on their experience and consideration of other properties. Again the two researchers that carried out the data collation ensured that the ratings given for structural damage were consistent.

3.4 Summary of Observed Damage

This section illustrates the range of damage to residential buildings and property that suffered the affects of liquefaction and lateral spreading.

3.4.1 Superstructure

The following images illustrate the typical modes and levels of damage found during the house inspections to different elements

3.4.1.1 Exterior Claddings

The most common exterior damage was to brick claddings, shown in Figure 3.4. Cracks were either vertical, horizontal or diagonal, and almost always through the mortar joints between bricks. Cracking through a brick, as shown in Figure 3.4.(c) was very rare. Cracks were anything from hairline to more than 10-20mm in width. In some cases whole corners were separated, as shown in Figure 3.4.(d).



Figure 3.4. Damage to brick cladding, a) low, b) moderate, c) high and d) corner separation and collapse.

There was very little damage to weatherboard cladding, supporting the findings of Buchanan (2011). In a few cases there was separation between elements, a severe case of which is shown in Figure 3.5.(b) where the door frame has separated from the weatherboard wall.

Figure 3.5.(a) and (c) show damage to concrete block cladding overlain by stucco. Damage to this is similar to that for brick cladding, with cracking through the mortar joints between concrete blocks, causing similar cracking patterns in the stucco covering. These images show a severe level of damage, with some separation and collapse of the cladding. Minor damage to stucco cladding, as shown in Figure 3.5.(d), is seen as minor hairline to 5mm cracks in the stucco, often at the corners of openings such as door and window frames.

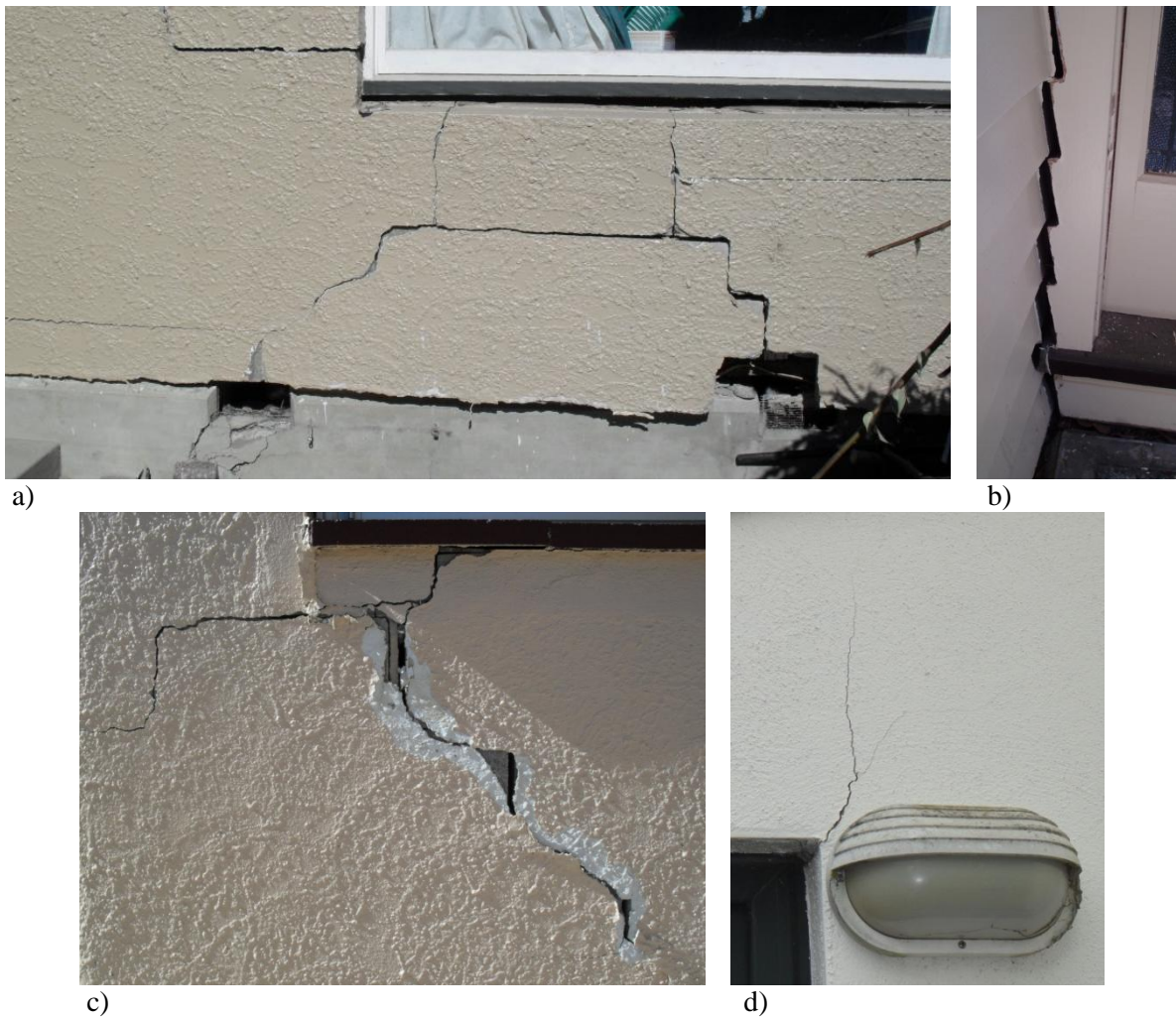


Figure 3.5. Damage to claddings, a) severe damage and partial collapse of stucco/concrete block cladding, b) high damage separation between weatherboard and door due to structural damage, c) moderate cracking in stucco/concrete block veneer and d) minor cracking in stucco veneer.

3.4.1.2 Interior Linings

Damage to interior linings involved cracking in the various gypsum and plaster board claddings found in residential houses in New Zealand. Minor cracking as shown in Figure 3.6 often occurred at joints in the plasterboard, or at wall openings such as the corners of door and window frames.

Higher levels of damage to interior claddings involved larger cracking and separation in plasterboard, as shown in Figure 3.7. Again this is more likely to occur at the corners of openings, where concentrations in stress are present.

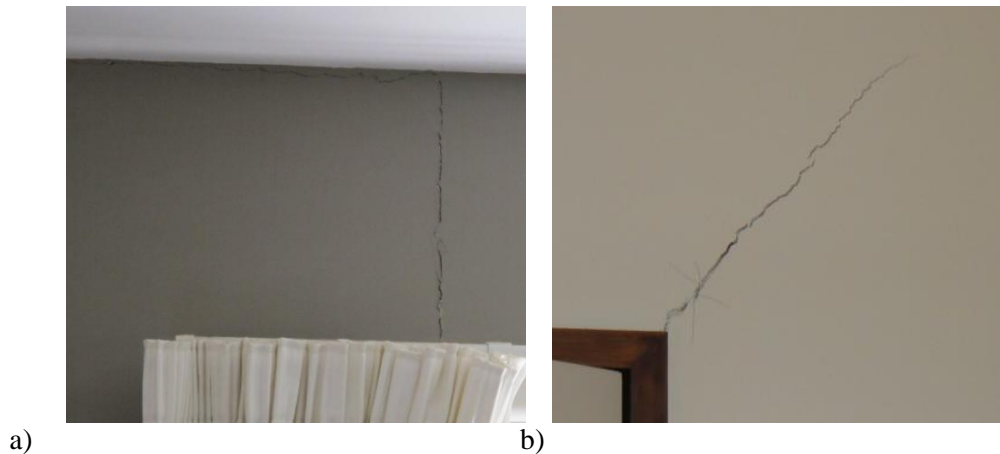


Figure 3.6. Interior lining damage, a) low damage vertical crack in plasterboard above window and b) low-moderate diagonal crack above door in plasterboard.



Figure 3.7. Interior lining damage, a) moderate-high diagonal crack in plasterboard above window, b) and c) high-severe diagonal cracks in plasterboard at window corners.

3.4.1.3 Roof

Damage to roofs and ceilings was generally not severe, except in the most extreme cases on lateral spreading. Typical exterior damage was the separation between the wall and roof interface as shown in Figure 3.8.(b). Interior damage was again seen as cracking in the gypsum and plaster board ceilings, again often at joints (Figure 3.8.(a) and Figure 3.9).

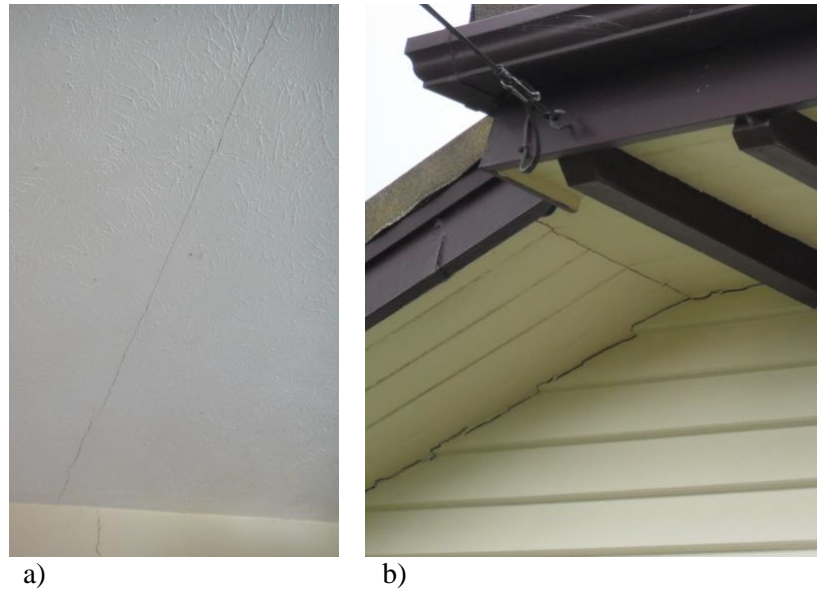


Figure 3.8. Ceiling and roof damage, a) low cracking at plasterboard joint in ceiling, b) low-moderate cracking and separation between weatherboard wall and roof.



Figure 3.9. Moderate-high damage to ceiling, cracking through plasterboard.

3.4.2 Foundations

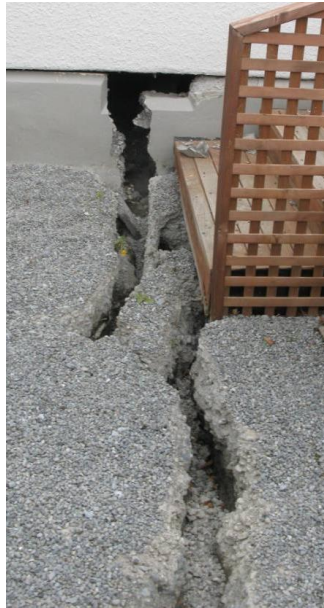
Damage to concrete perimeter foundations caused by liquefaction ranged from minor hairline and 1-2mm cracking, as shown in Figure 3.10.(a) to much larger cracks with anywhere from 50-100mm of separation and significant lateral offsets, as shown in Figure 3.10.(d) and (e). The most common place for cracking to occur in concrete perimeter foundations is at the air vents, as shown in Figure 3.10.(b), (c), (d) and (f). This is to be expected, as these are reduced sections with lower strength.



Figure 3.10. Cracking in concrete perimeter wall, a) low, b) and c) low-moderate, d) high, e) severe showing exposed reinforcing and f) multiple foundation cracks at air vents.

Lateral spreading caused the most severe damage to concrete perimeter foundations. As shown in Figure 3.11.(a) and (b), the perimeter is pulled apart by the ground movement, causing large cracks and extensive separation in the foundations. In extreme cases, lateral spreading can pull or push the concrete perimeter out completely from under the superstructure, as shown in Figure 3.11.(c) and (d). This can result in the superstructure sinking down inside the foundation causing severe structural damage.

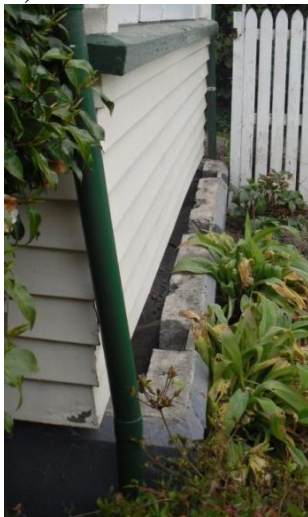
The most severe structural damage was generally seen where separation between the foundation and superstructure occurred. This was also observed by Buchanan et. al. (2011) and it appears that the integrity of the foundation-structure system is crucial to the overall performance of the building. No requirements were found in the housing standards prior to 1960 for a certain level of connection between the foundation and superstructure, meaning that many houses built before this time had inadequate connection between structural elements, making them vulnerable to significant structural damage.



a)



b)



c)



d)

Figure 3.11. Lateral spreading damage to concrete perimeter walls, a) and b) spreading cracks in ground causing cracking and large separation of perimeter, c) perimeter pulled out from under house by lateral spreading and d) perimeter pulled out from house on two sides and cracked by lateral spreading.



Figure 3.12. All piles under floor failed due to tilting caused by permanent lateral displacement of foundation relative to superstructure.

Failures were also seen in some short piers that support the floor in concrete perimeter foundations. Where significant lateral movement of the foundation or superstructure occurred, piers attached to the sub-floor structure could be pulled over due to their short depth of embedment (Figure 3.12).



Figure 3.13. Cracking in slab-on-grade floors, a) minor crack, b) and c) moderate cracking, d) moderate separation between slab sections, e) and f) moderate-high cracking and separation in slab.

Cracking and separation was seen in some concrete floor slabs, although most floor slabs were not visible as they were covered by carpet or other linings. Damage ranged from minor hairline cracking (Figure 3.13.(a)) to moderate to severe cracking and separation between slab sections as shown in

Figure 3.13.(c), (d), (e) and (f) and Figure 3.14.(a) and (b). There were a small number of vertical offsets in cracks, as shown in Figure 3.13.(f) and Figure 3.14.(b). The slab cracking found did not seem to follow any typical patterns as to where or how the cracking occurred.

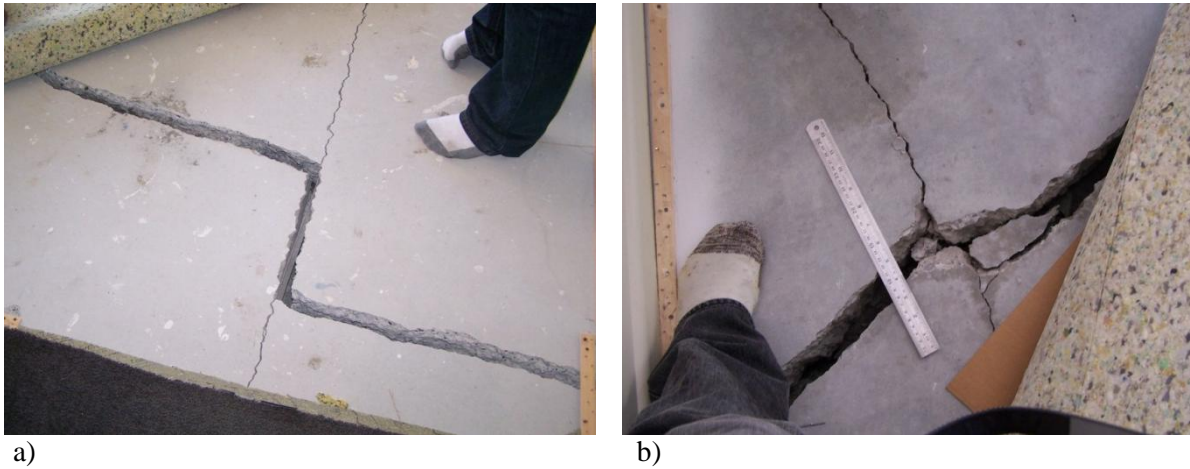


Figure 3.14. High-severe damage to slabs involving cracking and separation, shown in both a) and b).

3.4.3 Structure

The most common visible structural damage was racking of walls. Other forms of structural damage such as nail slip and damage to the timber framing were generally not visible in the houses inspected. Racking was mostly observed as mis-alignment in door and window frames, and was seen in both interior and exterior walls, as shown in Figure 3.15 and Figure 3.16. Racking of walls in houses with slab foundations was more uniform, as shown in Figure 3.16.(a), due to uniform differential tilting of the slab while the superstructure tries to remain upright. Racking in houses with concrete perimeter foundations tended to be less uniform, as shown in Figure 3.15.(a) and (b) and Figure 3.16.(b), due to the more flexible deformation modes of this type of foundation.

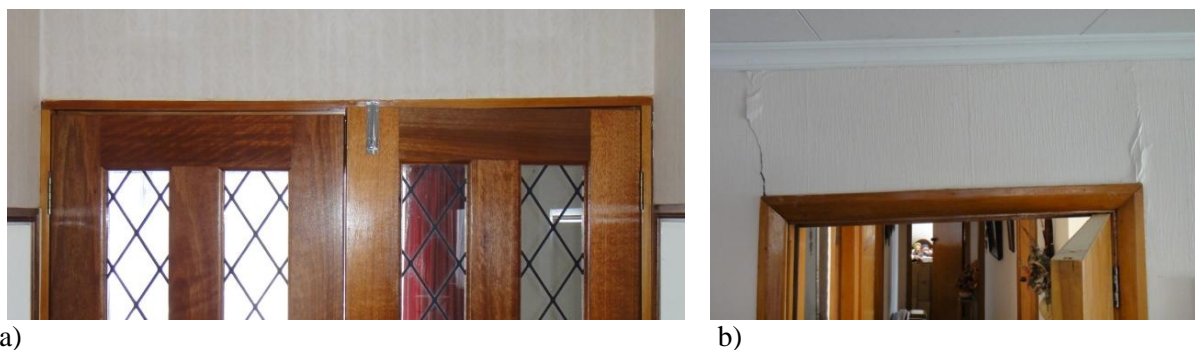


Figure 3.15. Structural damage, a) and b) racking of internal doors due to differential settlement

Another common type of structural damage occurred at the interface between external walls and the foundation. This was manifested in a number of ways, such as cracking along the interface, vertical separations and pushing out or misalignment of the wall and foundation, as shown in Figure 3.17.(b).

In a few extreme cases there was total separation of whole sections of the structure as shown in Figure 3.17.(a), where both the superstructure and foundation were ripped apart.

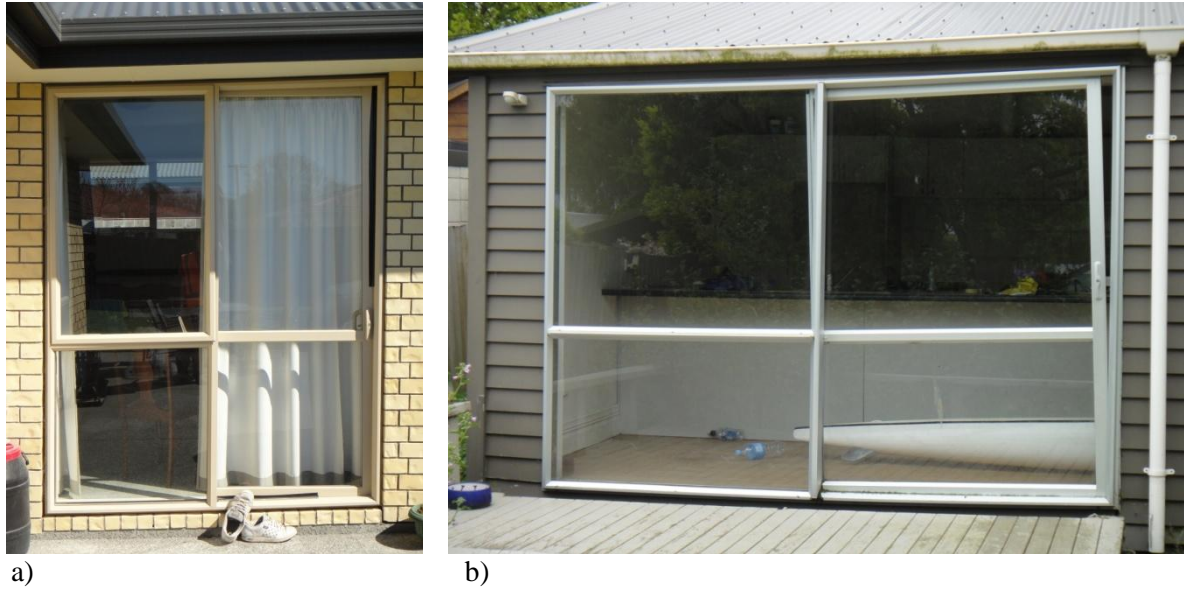


Figure 3.16. Structural damage, racking of external doors due to differential settlement, a) slab-on-grade foundation showing constant racking and b) concrete perimeter showing differential racking.



Figure 3.17. Structural damage, a) differential settlement causing separation between sections with different foundations and b) push-out of wall over concrete perimeter foundation wall causing collapse of bricks.

3.4.4 Surrounding Property

The most common damage to surrounding property was cracking in concrete paving. This was more common than cracking in concrete slab foundations, as paving is weaker and more easily affected by liquefaction, as it is cast straight onto the ground. Minor and moderate damage consisted of small cracks and low levels of separation between slab sections, shown in Figure 3.18.(a) and (b). More severe damage involved larger cracks and significant separation and break-up of paving, along with differential tilting and settlement, as shown in Figure 3.18.(c), (d) and (e).



a)



b)



c)



d)



e)

Figure 3.18. Damage to concrete paving, a) low, b) low-moderate cracking, c) moderate, d) high and e) severe cracking, break-up, differential settlement and separation.

Lateral spreading also produced severe damage to concrete paving, causing large cracking, differential tilting and separation as shown in Figure 3.19.(a), (b) and (c).

There was less damage to fencing but some was caused by liquefaction ejecta pushing up and distorting the fencing or lateral spreading ripping it apart (Figure 3.19.(d) and (e)).



Figure 3.19. Damage to surroundings, a), b) and c) severe damage to paving and ground due to large lateral spreading cracks, f) moderate damage to fence and g) high-severe damage to fence.

3.5 Data Scrutiny and Analysis

3.5.1 Data Quality Assurance and Censoring

The data collected during the summer programme following the 4th September 2010 earthquake was scrutinised to produce a data set suitable for analysis for this research.

The first step taken was to remove from the data set all the inspected properties that did not provide useful information for analysis. These included:

- All inspected buildings that weren't residential houses. These mostly came from reconnaissance inspections carried out immediately following the 4th September 2010 earthquake. During this period, all buildings severely affected by liquefaction in residential areas were being inspected. This meant that some shops and larger buildings were included.
- All inspected houses with incomplete information. These were houses where there was data missing either on the house characteristics, such as foundation or structural type, which would prevent them from being properly characterised. Proper characterisation is essential, as one of the aims of the analysis was to examine which design characteristics of houses may have affected their performance. Or those missing data on the damage suffered.
- Houses with unique or uncharacteristic features that could not be easily grouped. The decision was made to confine the data set to houses that had the most common characteristics, as their larger numbers allowed a more robust comparison.
 - o Foundation types were confined to either concrete perimeter with short piers, or standard slab-on-grade floors.
 - o Only one or two-storey houses were used. Anything larger or in-between was discarded.
 - o Cladding materials, both for roofs and walls, were limited to those that could fit into either the 'heavy' or 'light' category.

These steps resulted in a total of 31 inspected properties being removed from the dataset, leaving 461 residential houses with either concrete perimeter or slab-on-grade foundations to analyse.

3.5.2 Data Organisation

The next step was to select the construction characteristics that had the most effect on structural performance, and organise the data by these.

3.5.2.1 Foundation Type

The data was organised by the two foundation types; concrete perimeter with short piers and slab-on-grade. All other types of foundation were excluded as discussed above. The type of foundation in a house is important as it significantly influences the modes in which the structure will deform, and how

it will be damaged. Buchanan et. al. (2011) suggested that various foundation types performed quite differently when subjected to liquefaction. As most New Zealand houses have a similar superstructure, with a light timber frame and different cladding materials, the superstructures will generally behave the same way under inertial loading. As discussed in Chapter 2, damage due to shaking effects is much less than that caused by liquefaction. As liquefaction loads are displacement related, and coming from the ground, the foundation is the structural element with the biggest influence on the deformation and damage modes that occur in response to the liquefaction loads.

3.5.2.2 House Age

House age defines what construction standard the building should have been constructed to, and so gives some idea as to the strength of the foundation. Property age for each inspected house was provided by Property IQ. Using these ages, the houses were grouped into two categories, pre-standard and post-standard, as these represent the biggest changes in typical construction details for both concrete perimeter and slab-on-grade foundations. For concrete perimeter this cut-off is NZSS95:1935 when the first building standards were brought in. For slab-on-grade the cut-off is NZS3604:1981, as this was the first time prescribed construction details for residential house slabs were included in the standards.

3.5.2.3 Cladding Type

As was suggested by Buchanan and Newcombe (2010), heavier houses (i.e. those with heavy wall or roof claddings) seemed to suffer more damage and tended to settle more due to increased inertial loads. Buchanan et. al. (2011) also observed that heavier claddings were more easily damaged, due to stiffness incompatibilities between the cladding and the underlying structure it is attached to. To investigate this idea, houses were divided up based on their cladding materials into ‘heavy’ and ‘light’ types. Table 3.5 shows how the cladding materials were divided.

Table 3.5. Characterisation of cladding materials.

Cladding	Wall Claddings	Roof Claddings
Heavy weight	<ul style="list-style-type: none"> - Brick - Concrete block - Concrete - Stucco over brick/concrete block 	<ul style="list-style-type: none"> - Concrete tiles - Ceramic tiles - Slate
Light weight	<ul style="list-style-type: none"> - Weatherboard - Other timber - Iron - Light stucco 	<ul style="list-style-type: none"> - Light metal tiles - Iron - Light shingles

3.5.2.4 Number of Storeys

Due to the findings of Buchanan and Newcombe (2010) that heavier buildings suffered higher levels of damage, the data set was also divided between buildings with one and two storeys. A two storey building will be heavier than a one storey building with the same footprint, and so is likely to be damaged more severely based on these findings. Two-storey buildings also introduce the possibility of vertical structural irregularities, in weight, strength and stiffness and as was found by Beattie et. al. (2011), Buchanan et. al. (2011) and Liu and Beattie (2012), structural irregularities often mean higher levels of damage.

3.5.3 Addition of New Indices

The data collected on the extent of land damage (i.e. ‘liquefaction’ and ‘lateral spreading’ damage severity) was based on the visual observations of the various inspectors, which by nature is somewhat subjective. Because of this, it was decided to add new land damage indices to the data set from the EQC ProjectOrbit database in order to validate the land damage data collected in the inspections.

3.5.3.1 Vertical Ground Surface Movements

Vertical ground surface movements are the vertical elevation changes between LiDAR sets taken before and after the 4th September 2010 event that approximate the vertical ground movement during the earthquake. These data are limited by the lower accuracy of the pre-earthquake LiDAR data. The local vertical movements were calculated as the difference between the ‘observed’ elevation differences and the associated tectonic models. The data are provided as a coloured map in Google Earth, with different colours representing various sized increments in increased or decreased ground level from the +1.0 to +1.5m bin to the -1.0 to -1.5m bin. The colour grid is quite fine, with multiple colours often present within the footprint of one house. Because of this, the total range of values over the approximate footprint of each house was recorded, and an average taken to get a single value for the settlement. This is illustrated in Figure 3.20 where for AS104 the range taken would be -0.1m to -0.4m, with an average settlement of -0.25m. Similarly for AS105, the range would be +0.1m to -0.2m, an average of -0.05m.

Table 3.6. Settlement ranges used for this research.

Rating	Name	Settlement Range
0	None	$x < 0.05\text{m}$
1	Low	$0.05\text{m} \leq x < 0.1\text{m}$
2	Moderate	$0.1\text{m} \leq x < 0.2\text{m}$
3	High	$0.2\text{m} \leq x$

To allow a more direct comparison to the other data indices for the inspected houses, and to remove the inaccuracies in the LiDAR data and in reading the maps, the settlements were grouped into four broader categories. These are shown in Table 3.6.

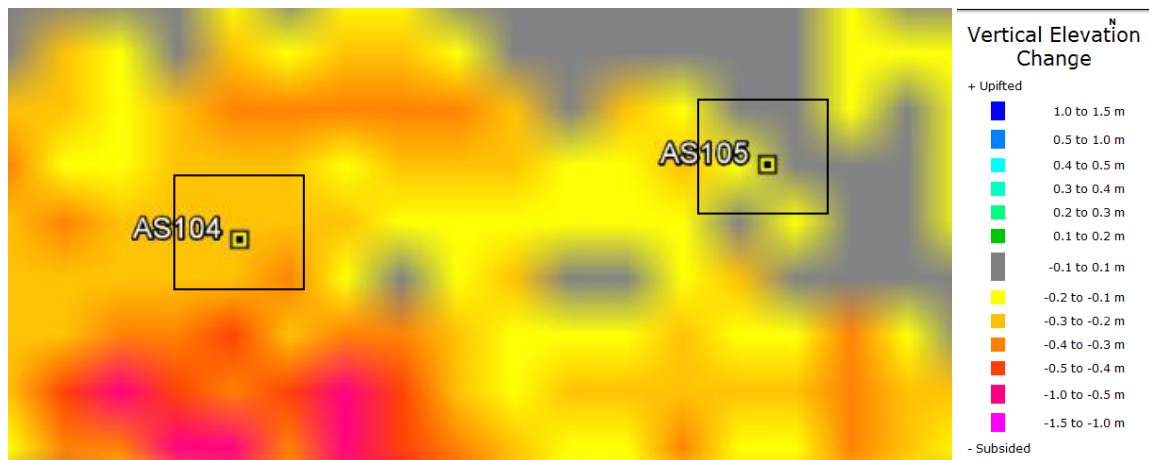


Figure 3.20. Distribution of ground settlement for two inspected houses AS104 and AS105, showing approximate size of house footprint and map key (Canterbury Geotechnical Database, 2012).

3.5.3.2 Horizontal Ground Movements

These are the local horizontal ground surface movements between LiDAR sets taken before and after the 4th September 2010 event that approximate lateral ground movements during the earthquake. The horizontal movements were calculated from the differences between subsequent LiDAR data sets on a 4m grid, and averaged to give movements in a 56m grid. The horizontal movements are affected by elevation errors in the LiDAR data, and by localised changes such as building demolitions, vegetation or excavation. Like the vertical movements, these are only as accurate as the pre-earthquake LiDAR data set, which is less accurate than the post-earthquake dataset. The overall movements were calibrated against surveyed movements of the benchmark networks of the Christchurch City Council, Land Information New Zealand and Environment Canterbury. The local movements were calculated as the difference between observed movements and tectonic models. The data is presented as vector arrows in the 56m grid in Google Earth, where an arrow 56m in length (i.e. between two adjacent grid points) represents 1m of movement in the direction indicated. Figure 3.21 shows the local horizontal movement arrows used to determine the lateral spreading of houses AS104 and AS105. As there is an arrow close to each house, little interpolation is needed, and values of 0.3m NNE and 0.4m NNE were used for AS104 and AS105 respectively.

As for settlement, the lateral spreading values were grouped into four more general categories, to allow comparison with the other data indices for the inspected houses, and to account for the inaccuracies inherent in the LiDAR data, and in reading the data to give values for each house. These are shown in Table 3.7.



Figure 3.21. Local horizontal movement arrows (in green) indicating movement for houses AS104 and AS105 (Canterbury Geotechnical Database, 2012).

Table 3.7. Lateral Spreading ranges used for this research.

Rating	Name	Lateral Spreading Range
0	None	$x < 0.1\text{m}$
1	Low	$0.1\text{m} \leq x < 0.3\text{m}$
2	Moderate	$0.3\text{m} \leq x < 0.5\text{m}$
3	High	$0.5\text{m} \leq x$

3.5.4 Analysis

Following the organisation of the data into the different construction characteristics of foundation type, house age, cladding type and number of storeys, the following graphs were produced to compare the different damage indices with each other.

- Liquefaction severity vs. Foundation damage
- Settlement (from LiDAR) vs. Foundation damage
- Lateral spreading (from LiDAR) vs. Foundation damage
- Foundation damage vs. Superstructure damage

A number of different versions of these graphs were made with the data organised by the following factors:

- The three different suburbs where buildings were inspected
- Different foundation types
- Pre and post-standard construction
- Heavy and light claddings
- One and two storey buildings
- Combinations of these

This allowed analysis of the trends for the different damage indices and how they affected the performance of houses, and what construction or other factors may have influenced these trends.

3.6 Results

This section presents the results obtained from the analysis of the different damage indices and construction details discussed in Section 3.5. A number of graphs have been selected that best illustrate the range of results from specific to typical that were obtained through the analysis of the data. For other graphs that are not shown here, see Appendix E.

3.6.1 Foundation Damage

The foundation damage caused by the various effects of liquefaction is of particular interest to this research. As the foundation is directly influenced by liquefaction effects, the general trends in this data will be examined first. The ‘foundation damage’ values used in this section were those recorded by the summer research teams based on the subjective inspection ratings criteria outlined in Section 3.3.2.2.

3.6.1.1 Inspected Liquefaction Rating

The inspected liquefaction rating was the final rating for liquefaction on the four-point scale of ‘none’ to ‘high’ that was collected by the summer research team. It provides simple but valuable insight into the overall trend in foundation damage in houses affected by liquefaction effects.

Figure 3.22 illustrates the distribution of foundation damage to houses that suffered differing levels of liquefaction. It shows that for an increase in liquefaction severity, the proportion of houses suffering higher damage to their foundations increases. For no liquefaction, there were no foundations with high damage, and less than 10% with moderate damage. This is compared to high liquefaction, where less than 10% of houses suffered no foundation damage, with a much higher proportion having high damage. The trend shown in Figure 3.22 is supported by Figure 3.23 which shows the same data but plotted the opposite way, showing the distribution of liquefaction severity at each level of foundation damage. This shows that foundations with higher damage were more likely to have suffered higher levels of liquefaction also.

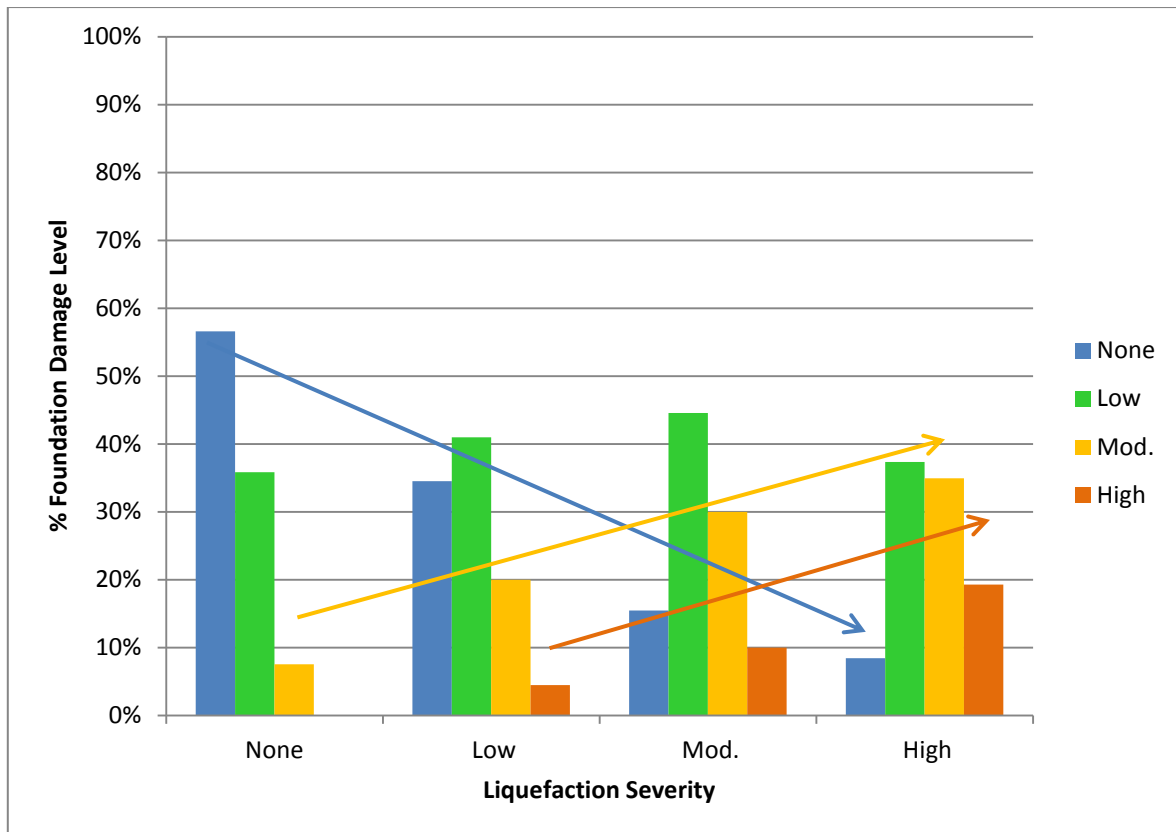


Figure 3.22. Distribution of foundation damage level in percentage for differing levels of liquefaction severity for all houses.

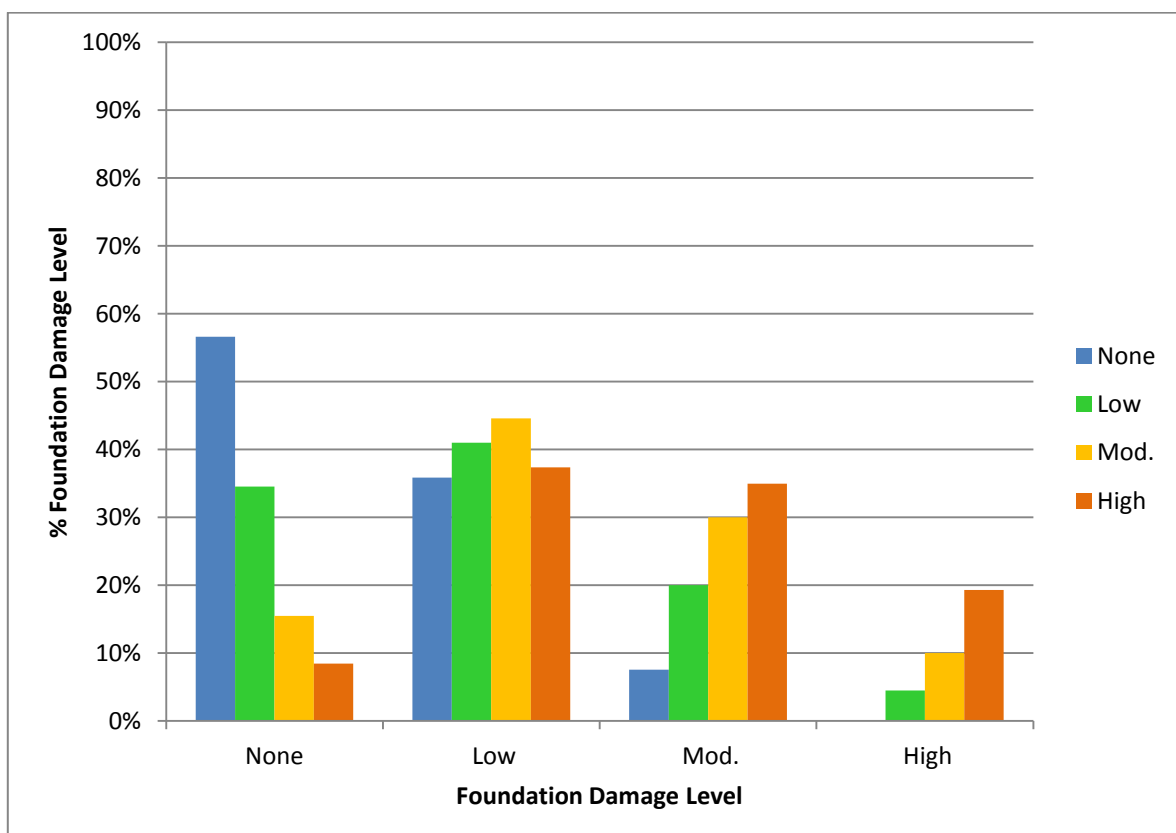


Figure 3.23. Distribution of liquefaction severity experienced for houses at each level of foundation damage.

3.6.1.2 LiDAR Ground Settlement

The settlement values gathered from the Canterbury Geotechnical Database support the trend found in the data for liquefaction severity as judged by the summer research team. This is to be expected, as the two land damage indices are directly related. A higher level of liquefaction is likely to result in higher levels of settlement. More severe liquefaction damage indicates the presence of thicker liquefied layers or looser soils, or both. This leaves more room for post-liquefaction reconsolidation settlement, which is the largest contributor to overall settlement in the free field, along with the loss of soil volume due to sand ejecta. Figure 3.24 shows that the proportion of foundations with no damage steadily decreases as the overall global ground settlement increases, while the proportion of foundations suffering high damage increases. For ground settlement over 200mm, more than 25% of all houses inspected suffered high levels of damage to their foundations, with more than 55% suffering at least moderate foundation damage.

Again, as for liquefaction severity, this trend is supported by Figure 3.25 which shows the same data as Figure 3.24 but plotted the opposite way, showing the distribution of ground settlement for foundations at each damage level. Foundations suffering no damage are more likely to have experienced low levels of ground settlement, while foundations with high damage are much more likely to have experienced ground settlement over 200mm.

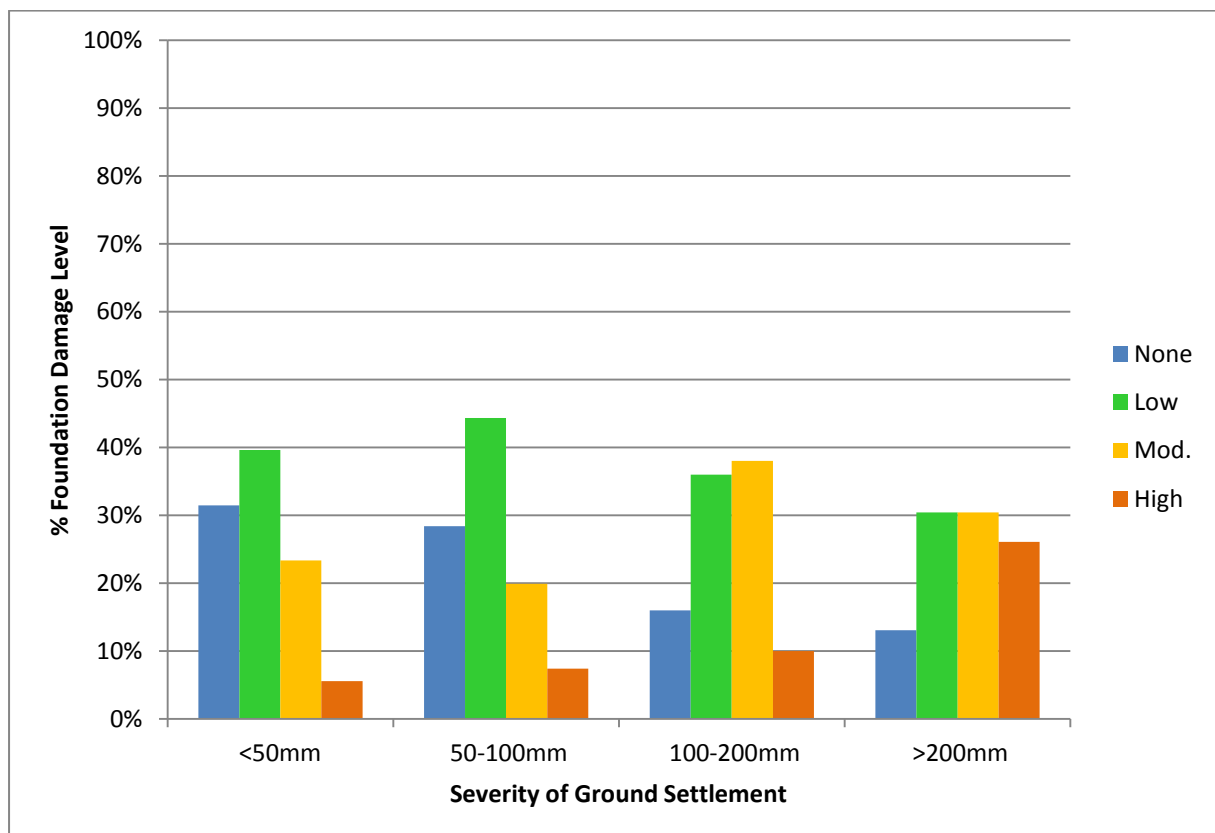


Figure 3.24. Distribution of foundation damage level in percentage for differing degrees of ground settlement for all inspected buildings.

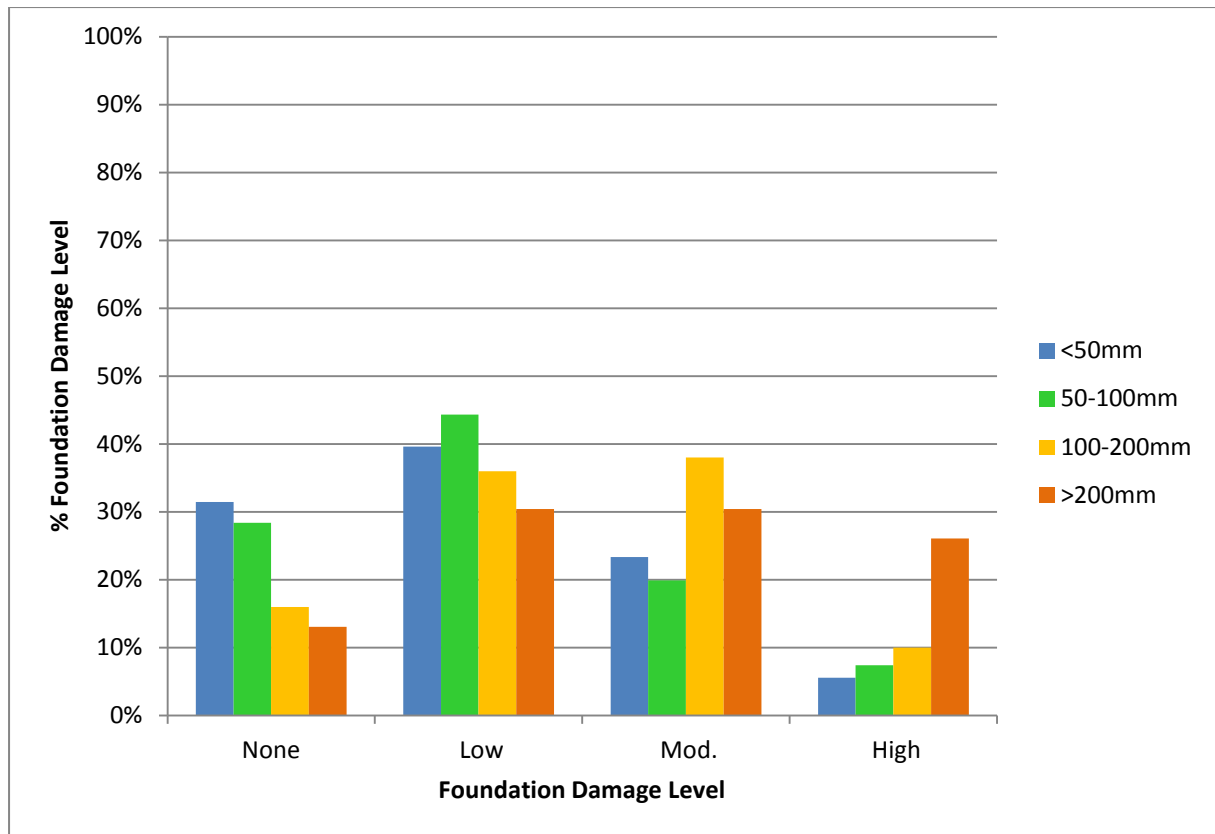


Figure 3.25. Distribution of ground settlement experienced for houses at each level of foundation damage.

3.6.1.3 LiDAR Lateral Spreading

The distribution in percentage of foundation damage with different levels of lateral spreading from the LiDAR data is shown in Figure 3.26. For lateral spreading over 100mm, the data shows the same trends that are seen in the Liquefaction and Ground Settlement data. As the level of lateral spreading increases, the proportion of foundations suffering no damage reduces, and the foundations suffering moderate and high damage generally increases. This trend is expected, as lateral spreading puts large loads on structures, causing rupture and pull-apart of the foundations which is transferred to the superstructure resulting in significant damage.

The distribution of foundation damage for <100mm lateral spreading must be ignored, as there were less than 10 houses inspected in total that suffered less than 100mm of lateral spreading. This is shown in Appendix E:3, Figure A. 42, the equivalent graph for Figure 3.26 but distributed by numbers. This was due to the fact that all properties surveyed were in the worst-hit areas of Christchurch following the 4th September 2010 earthquake, and were all close to rivers where significant lateral spreading occurred which was unrepresentative of the overall lateral spreading that occurred in the earthquake. This means that the percentage distribution for the <100mm bin in Figure 3.26 cannot be used, as the small number of data points gives unrealistic results.

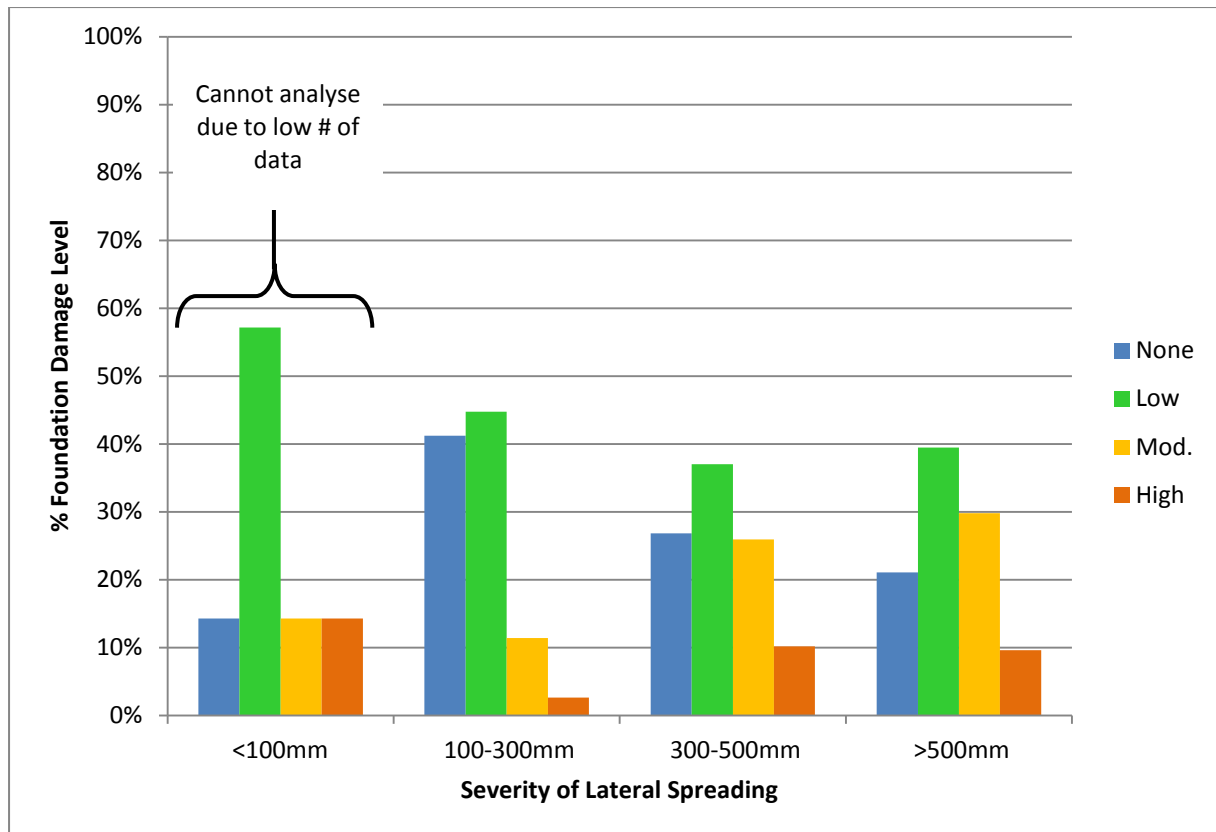


Figure 3.26. Distribution of foundation damage level in percentage for differing degrees of lateral ground movement for all inspected buildings.

3.6.2 Superstructure Damage

Superstructure damage to houses is the next step in the chain of damage caused by liquefaction and its associated effects. If the foundation is damaged, by cracking, tilting or settlement, this in turn affects the superstructure it is supporting. As discussed in Chapter 2, Section 2.3, the nature of liquefaction damage means that loads are displacement-related rather than the inertial loads that occur during strong shaking in an earthquake. Residential timber-frame houses of the type used in Christchurch and New Zealand are flexible, but cannot always accommodate the large loads caused by differential displacements that result from liquefaction-related settlement.

Figure 3.27 shows that the general trend in the distribution of superstructure damage is as expected. As the level of foundation damage increases, the percentage of houses suffering no damage to their superstructures decreases. In addition, the proportion of superstructures suffering moderate or high damage increases with the level of foundation damage, which is also to be expected. This is further supported by Figure 3.28, which exhibits the same general trends. It shows the same superstructure damage data for all houses, but against liquefaction severity level rather than foundation damage. When no liquefaction damage was visible, there were no houses which suffered more than minor superstructure damage, and even at low levels of liquefaction there were very low numbers of houses with moderate and high superstructure damage. The two houses that suffered high damage at low liquefaction levels also had highly damaged foundations. This could be due to weak foundations or

other special circumstances. This is similar to Figure 3.27, where for foundations with none or low damage, there were no houses which suffered high levels of superstructure damage.

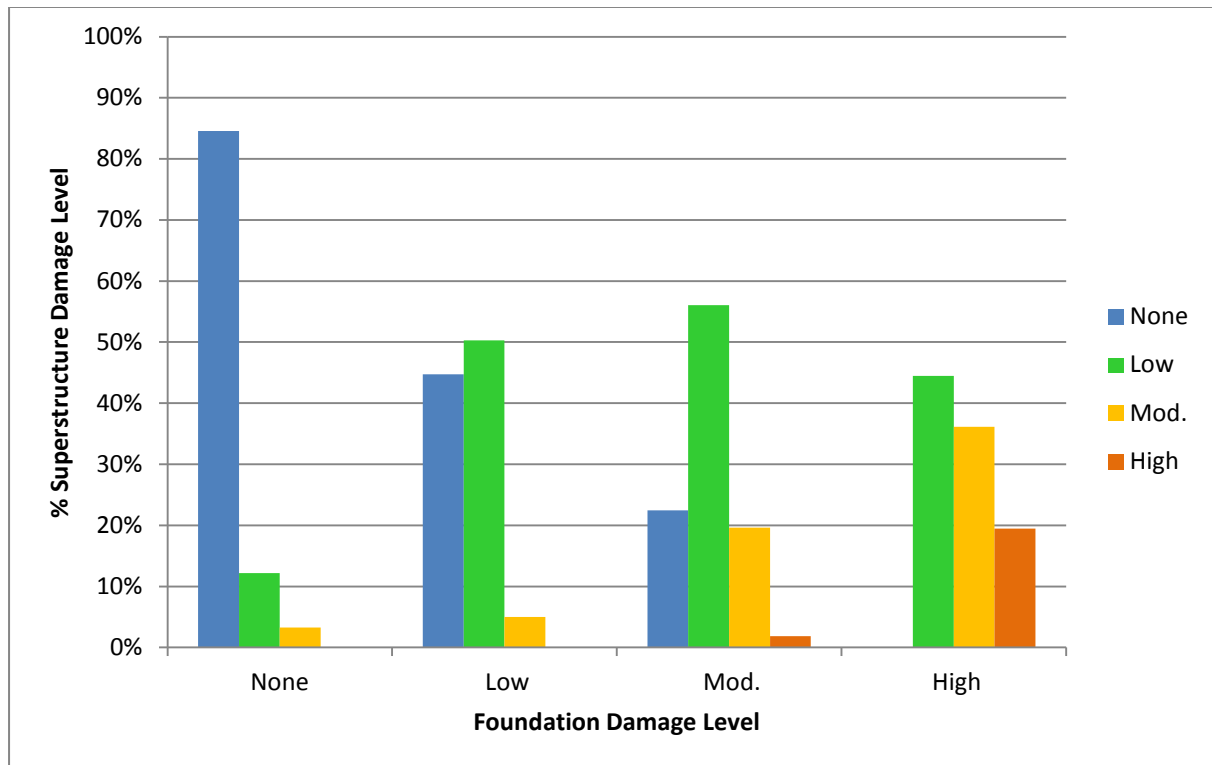


Figure 3.27. Distribution in percentage of damage to house superstructures for differing foundation damage levels for all houses.

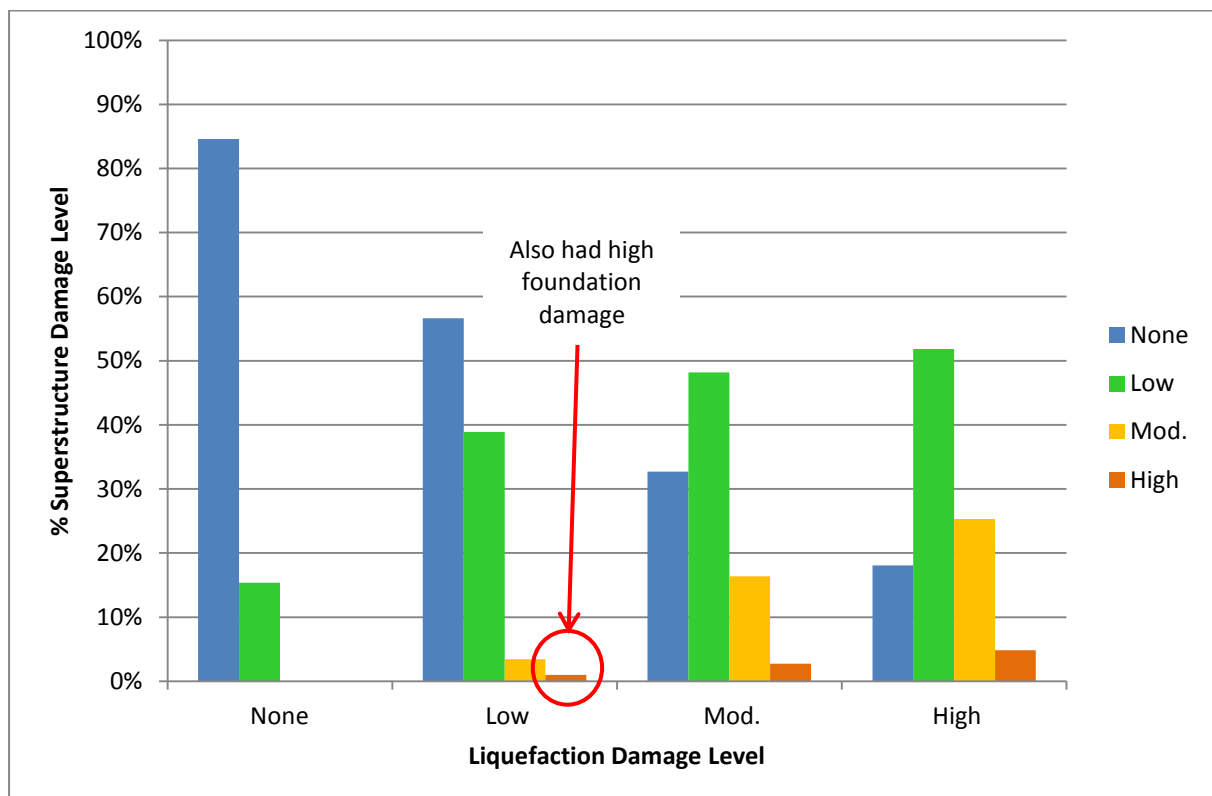


Figure 3.28. Distribution in percentage of damage to house superstructures for differing liquefaction damage levels for all houses.

3.6.3 Concrete Perimeter vs. Slab-on-Grade Foundations

Concrete perimeter footing with short pier supports, and concrete slab-on-grade foundations are the two most common types of foundations used in residential houses in Christchurch. Of the ~500 houses inspected by the summer research team, only 7 were excluded for foundations that did not fall under these two types.

It is useful to examine the behaviour of houses with these two different foundation types, as they are very different in their construction and load-bearing characteristics, and so are expected to behave very differently under seismic and liquefaction loads. This section explores the differences in their levels of performance, and associated trends. The differences between their modes of deformation and a more detailed analysis of the difference in performance of the two foundation types is explored further in Chapter 4.

3.6.3.1 Inspected Liquefaction Rating

When graphed against the inspected liquefaction rating, the performance of the two different foundation types was very similar. Figure 3.29 and Figure 3.30 for concrete perimeter and slab-on-grade foundations respectively both show the same trend that was seen in Section 6.1.1 for all houses with an increase in the proportion of moderate and highly damaged foundations as the severity of liquefaction increases.

Concrete perimeter foundations appear to have a larger proportion of instances of high damage than slab-on-grade foundations, with 14% and 25% of concrete perimeter foundations suffering high damage at moderate and high levels of liquefaction respectively, compared to 5% and 12% for slab-on-grade foundations. This would suggest that concrete perimeter foundations perform worse at higher liquefaction levels.

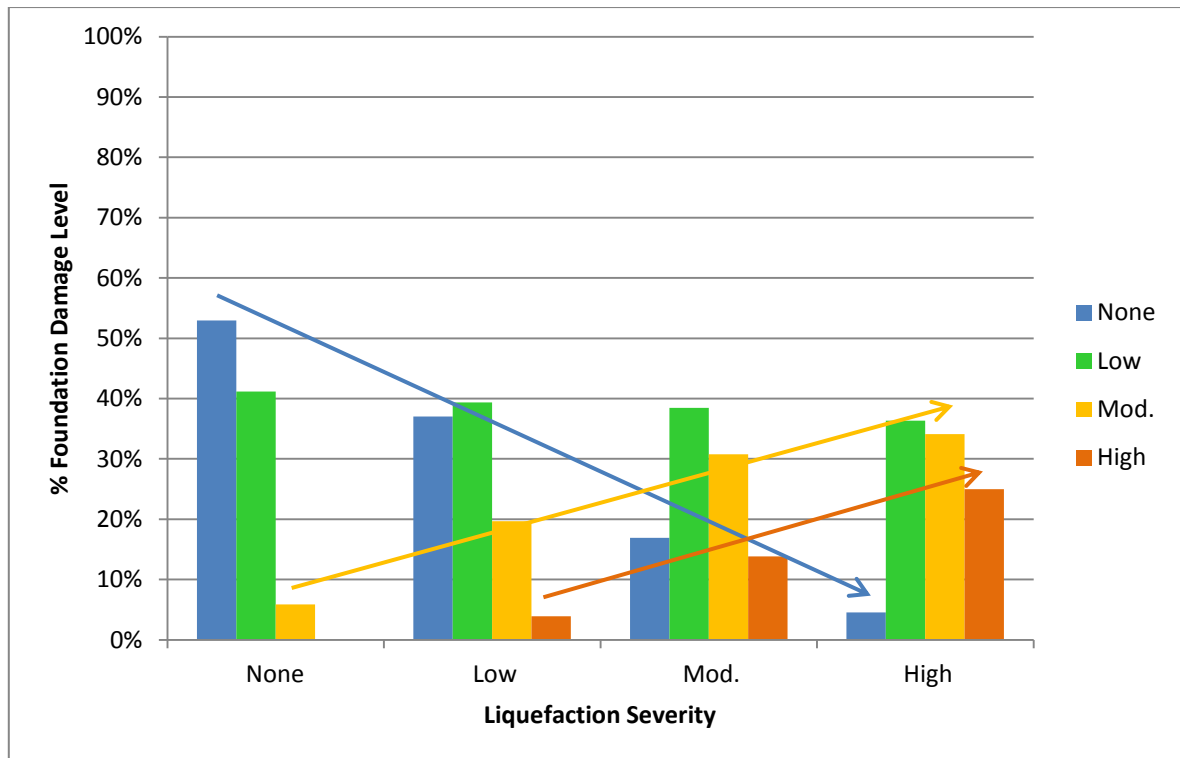


Figure 3.29. Distribution in percentage of foundation damage level for differing levels of liquefaction severity for concrete perimeter foundations.

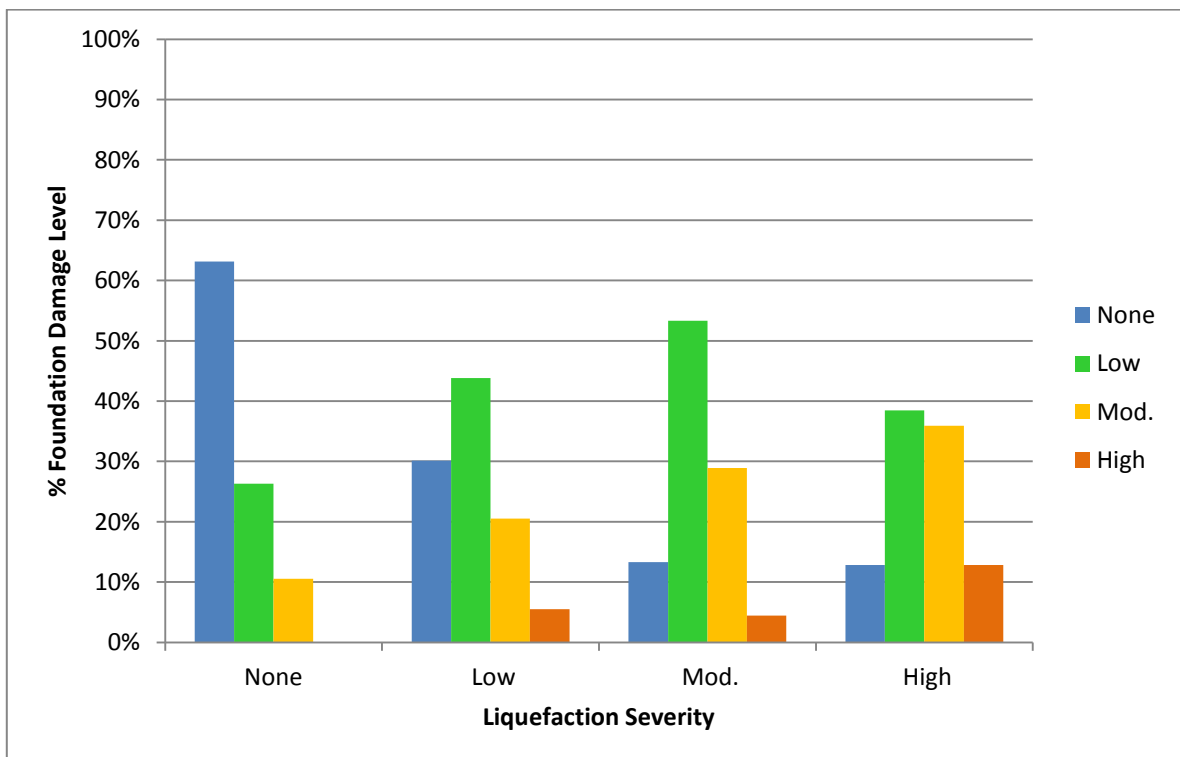


Figure 3.30. Distribution in percentage of foundation damage level for differing levels of liquefaction severity for slab-on-grade foundations.

3.6.3.2 LiDAR Ground Settlement

The plots for foundation damage at differing levels of settlement (Figure 3.31 and Figure 3.32) do not show any obvious trends for either foundation type, except that again, as for liquefaction severity, it appears a higher proportion of concrete perimeter foundations (Figure 3.31) suffered moderate or high

damage at high levels of settlement than for slab-on-grade foundations. However there were very few houses inspected that had high settlement (less than 20 for concrete perimeter and less than 10 for slab-on-grade) so these proportions are less certain than those for the lower settlement severities.

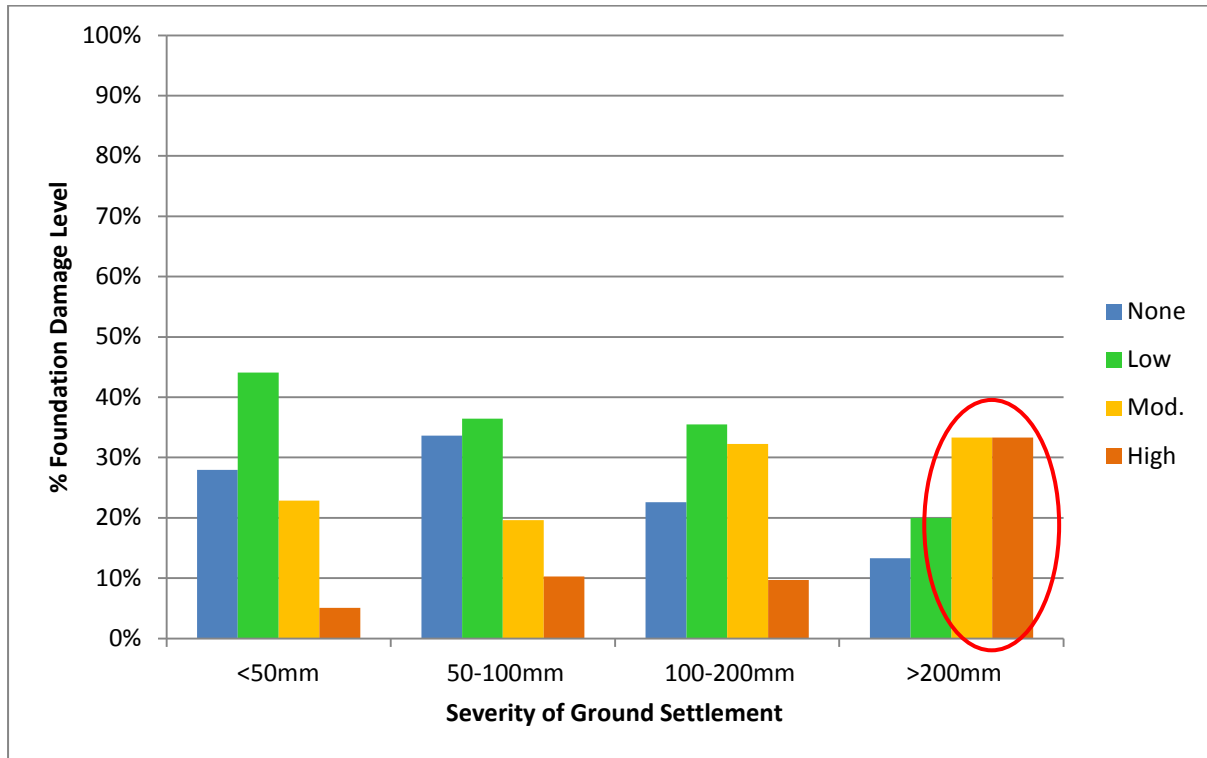


Figure 3.31. Distribution in percentage of foundation damage level for differing degrees of ground settlement for concrete perimeter foundations.

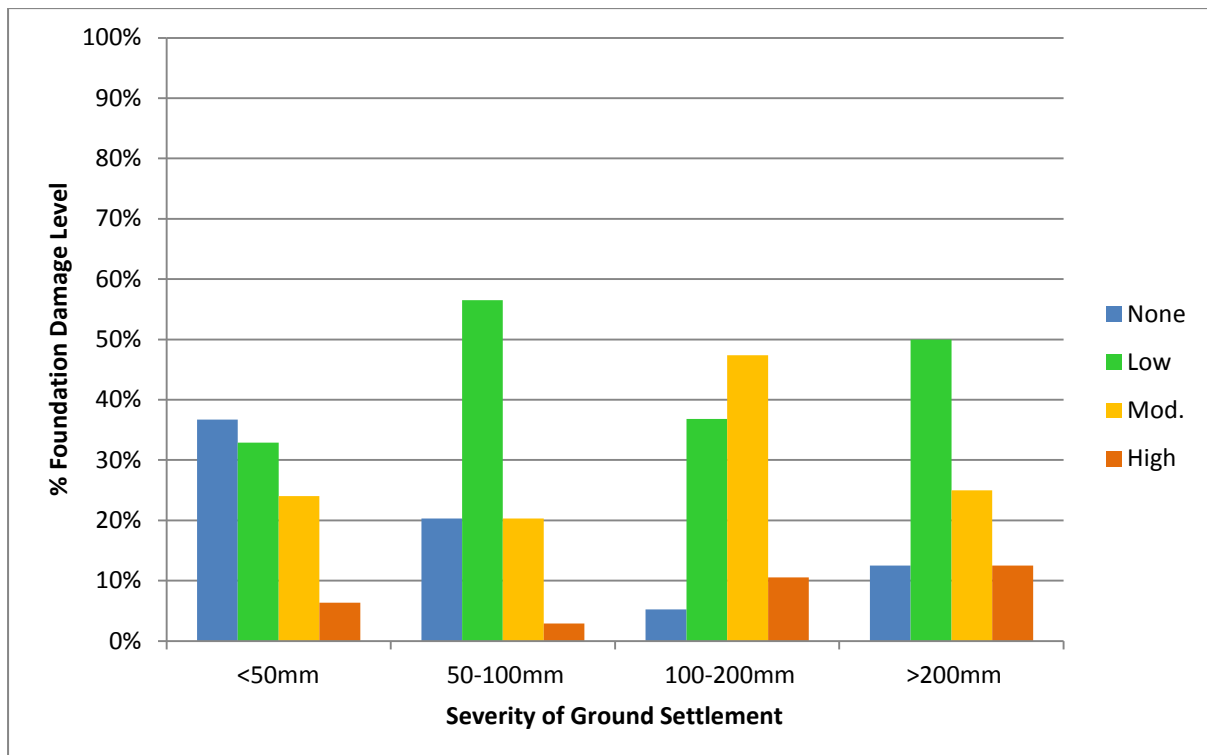


Figure 3.32. Distribution in percentage of foundation damage level for differing degrees of ground settlement for slab-on-grade foundations.

3.6.3.3 LiDAR Lateral Spreading

Figure 3.33 and Figure 3.34 show the distributions of foundation damage for differing levels of lateral spreading for concrete perimeter and slab-on-grade foundations respectively.

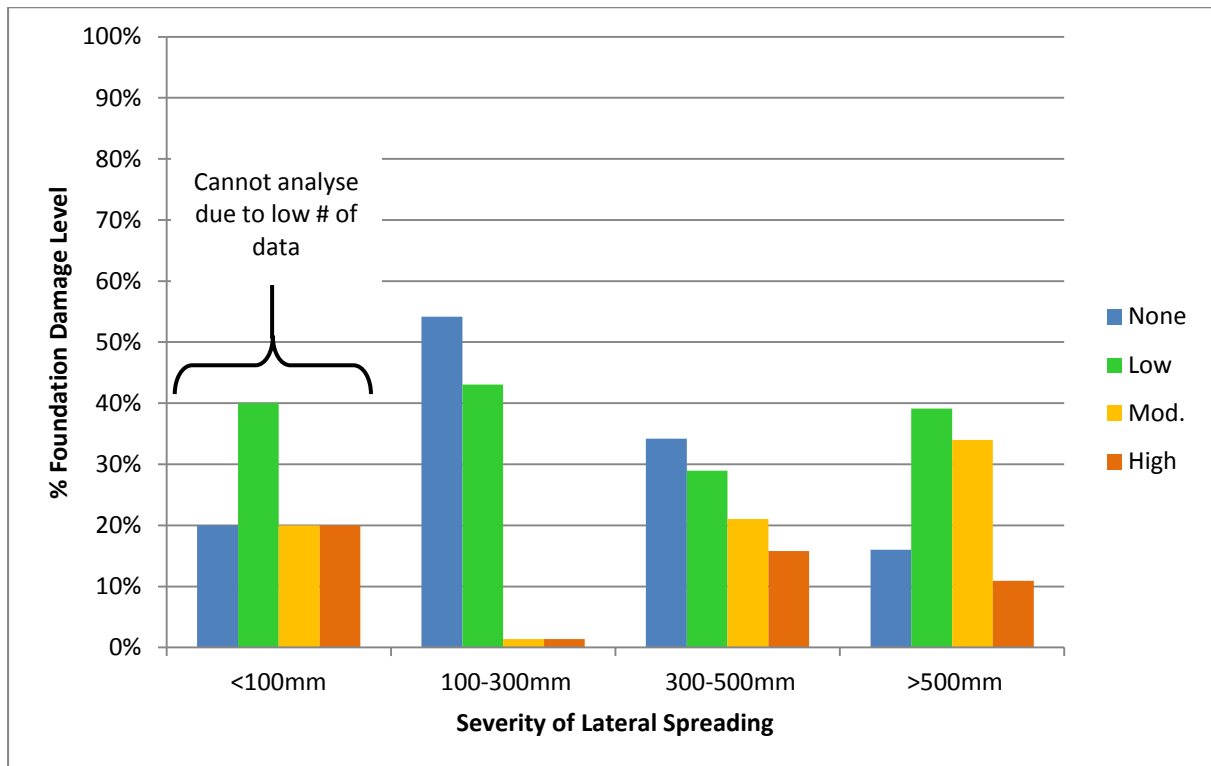


Figure 3.33. Distribution in percentage of foundation damage level for differing degrees of lateral spreading for concrete perimeter foundations.

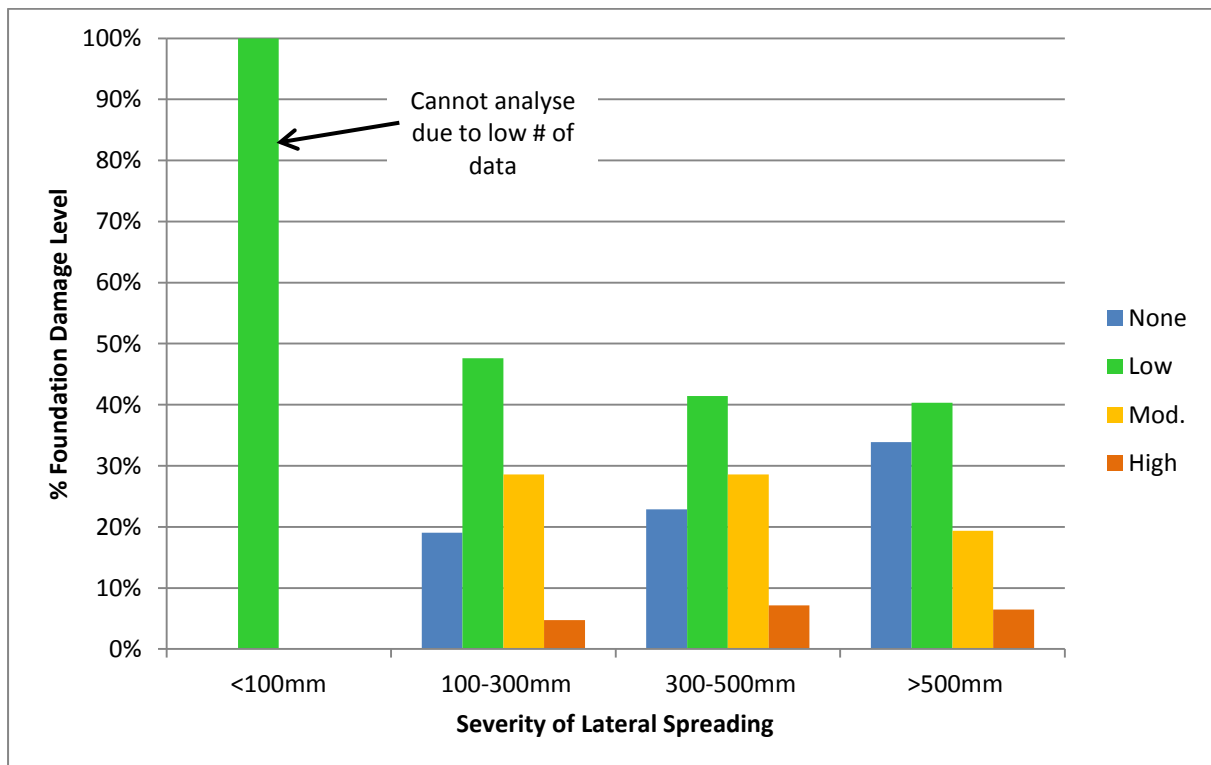


Figure 3.34. Distribution in percentage of foundation damage level for differing degrees of lateral spreading for slab-on-grade foundations.

When the two plots are compared, ignoring the <100mm lateral spreading bin due to the insufficient data, concrete perimeter foundations seem to have followed the overall trend of increasing level of foundation damage for increasing lateral spreading. As the level of lateral spreading in Figure 3.33 increases, the percentage of concrete perimeter foundations with no damage reduces, while foundations suffering moderate and high damage increases. The slab-on-grade data in Figure 3.34 is quite different. The overall distribution of foundation damage changes very little with increasing level of lateral spreading, suggesting slab-on-grade foundations perform better than concrete perimeter foundations under lateral spreading.

This is the only real conclusion that can be drawn from this data, due to the low number of inspected houses that experienced lower levels of lateral spreading. More data needs to be collected to further investigate the performance of foundations under lateral spreading.

3.6.3.4 Superstructure Damage

Figure 3.35 and Figure 3.36 show the distribution of superstructure damage at different levels of foundation damage for concrete perimeter and slab-on-grade foundations. They indicate very similar behaviour for the two different foundation types, except for high levels of foundation damage. At high levels of foundation damage, more than 60% of superstructures on concrete perimeter foundations suffered moderate or high damage, compared to less than 40% for slab-on-grade foundations. This suggests concrete perimeter foundations suffer worse damage at high levels, and affect the superstructure more due to their particular mode of deformation at high damage levels, and supports the trends seen in the graphs for foundation damage against inspected liquefaction rating and LiDAR settlement.

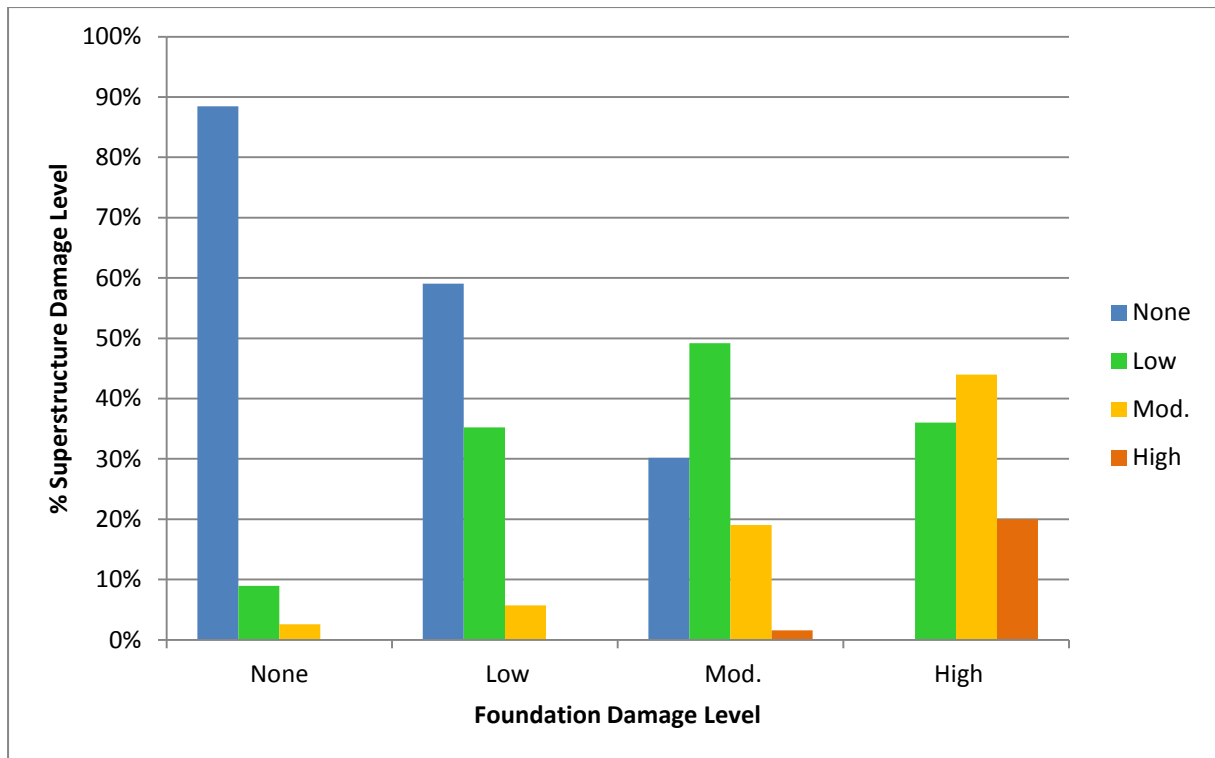


Figure 3.35. Distribution in percentage of superstructure damage for concrete perimeter foundation houses for different levels of foundation damage.

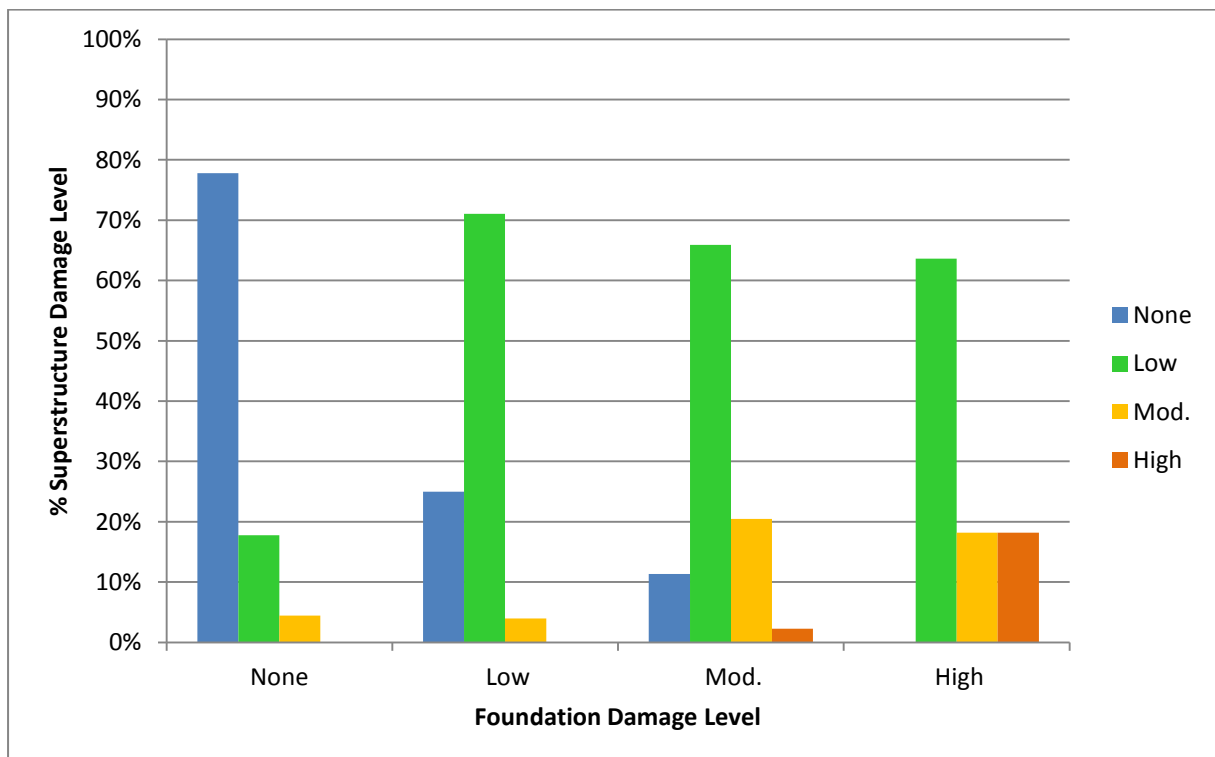


Figure 3.36. Distribution in percentage of superstructure damage for slab-on-grade foundation houses for different levels of foundation damage.

3.6.4 Pre-Standard vs. Post-Standard Construction

As is shown in Table 3.8, there were far fewer houses inspected of both foundation types that were constructed prior to the introduction of housing standards (i.e. pre-1930 for concrete perimeter foundations and pre-1980 for slab-on-grade foundations). This has limited the ability to draw any

definite conclusions from this data. However, there are some trends evident in the data, and these will be discussed in this section separately for the two different foundation types. This is due to their different behaviour and deformation modes when subjected to liquefaction, and also due to the 50 year difference in when their first standards were introduced, as the superstructures of houses changed somewhat between these times too.

Table 3.8. Number of inspected houses built pre and post-standards for each foundation type.

	Pre-Standard	Post-Standard
Concrete Perimeter	23	238
Slab-on-grade	36	141

3.6.4.1 Concrete Perimeter

The distributions of foundation damage for minor or significant liquefaction, shown in Figure 3.37 and Figure 3.38 indicate that post-standard foundations (Figure 3.38), performed better than pre-standard foundations (Figure 3.37), irrespective of the level of liquefaction. There were a higher percentage of pre-standard foundations that suffered high levels of damage, and less that suffered low foundation damage than post-standard foundations. This is to be expected, as pre-standard concrete perimeter foundations were constructed poorly compared to standardised construction. They often had no reinforcing, and used large items such as bricks or stones as fill in the concrete.

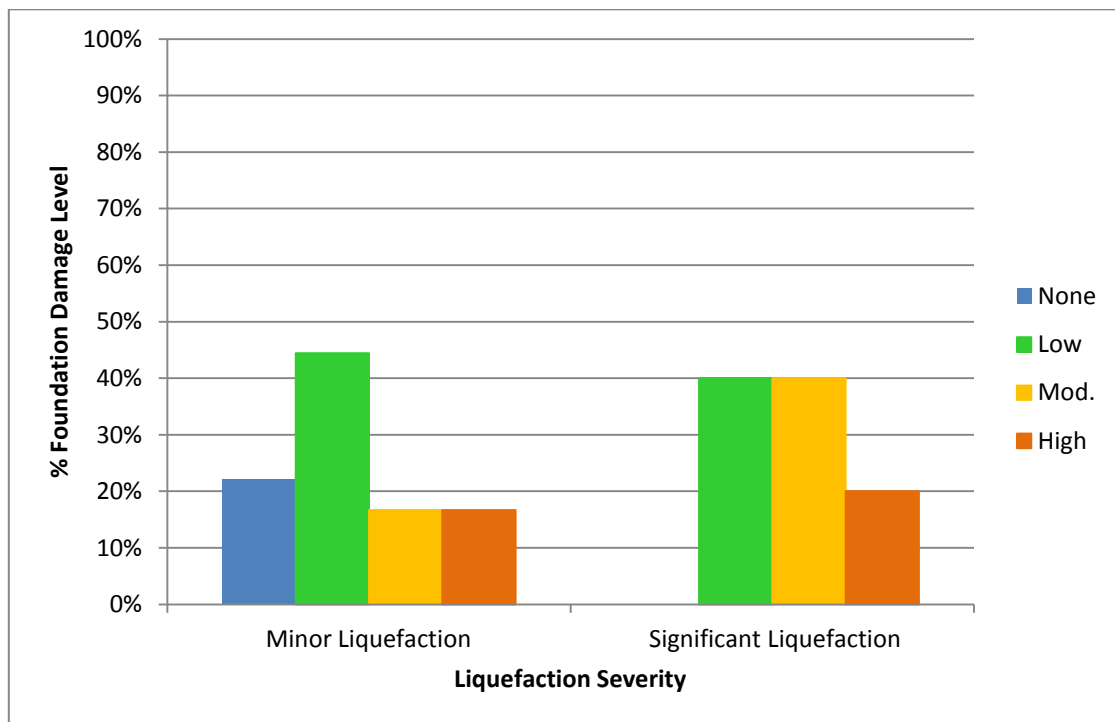


Figure 3.37. Distribution in percentage of foundation damage for pre-standard concrete perimeter foundations for minor or significant liquefaction.

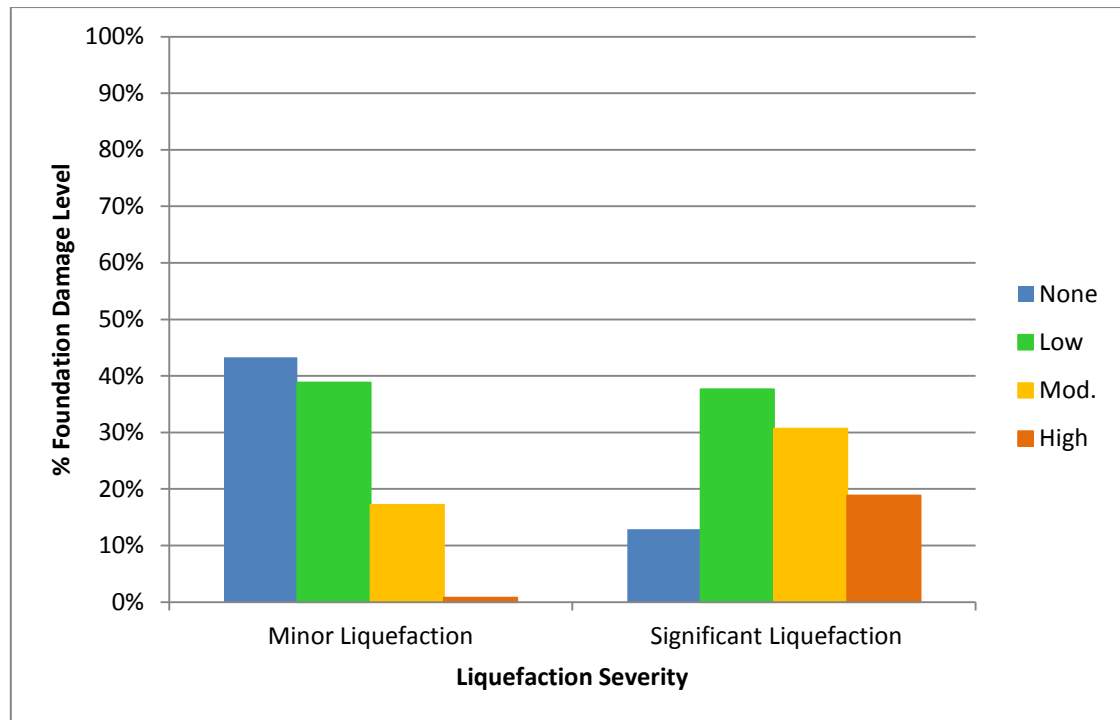


Figure 3.38. Distribution in percentage of foundation damage for post-standard concrete perimeter foundations for minor or significant liquefaction.

The trend for liquefaction is echoed by the equivalent plots for settlement (Figure 3.39 and Figure 3.40), but only for significant levels of settlement. Pre-standard foundations are more likely to suffer moderate or high damage than post-standard, again suggesting that pre-standard foundations are more easily damaged than standardised ones. For minor levels of settlement however the distributions are almost identical.

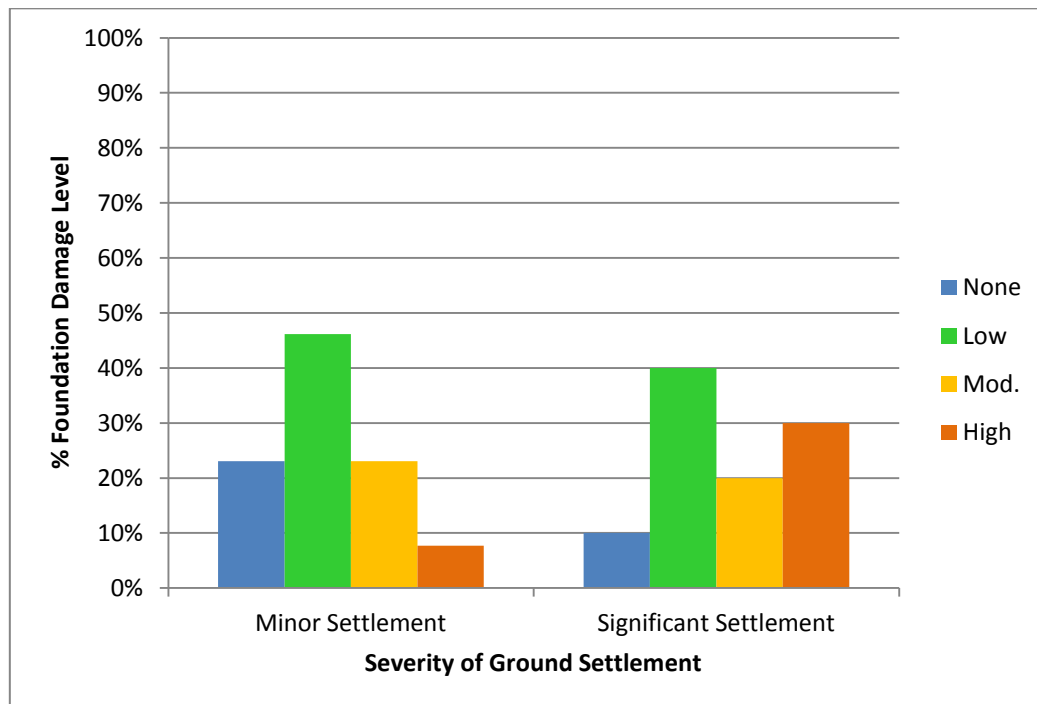


Figure 3.39. Distribution in percentage of foundation damage for pre-standard concrete perimeter foundations for minor or significant settlement.

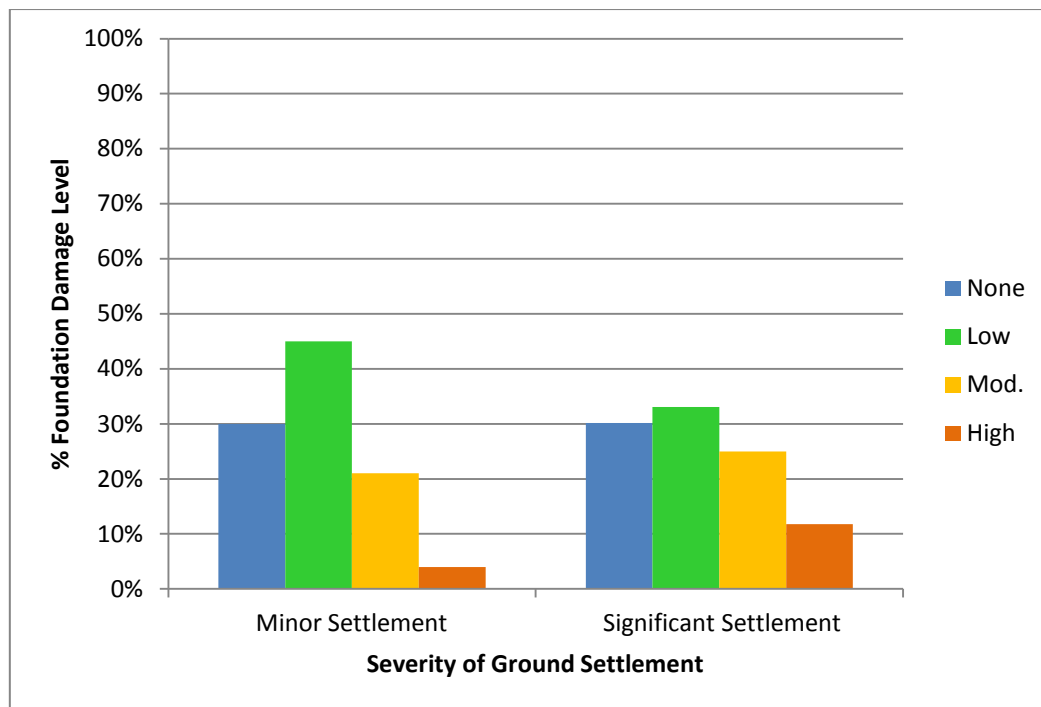


Figure 3.40. Distribution in percentage of foundation damage for post-standard concrete perimeter foundations for minor or significant settlement.

There was almost no difference in the performance of pre or post-standard concrete perimeter foundations subjected to lateral spreading. As shown in Figure 3.41, the distribution of minor and significant foundation damage at significant levels of lateral spreading is almost identical for pre and post-standard construction. This could be due to concrete perimeter foundations as a structural design in general performing badly under lateral spreading, despite their construction details, but is more likely that at higher levels of lateral spreading the stronger and more ductile standardised foundations do not add sufficient resistance to sustain the large loads imposed by the lateral spreading, so that foundations constructed in either period are just as badly damaged.

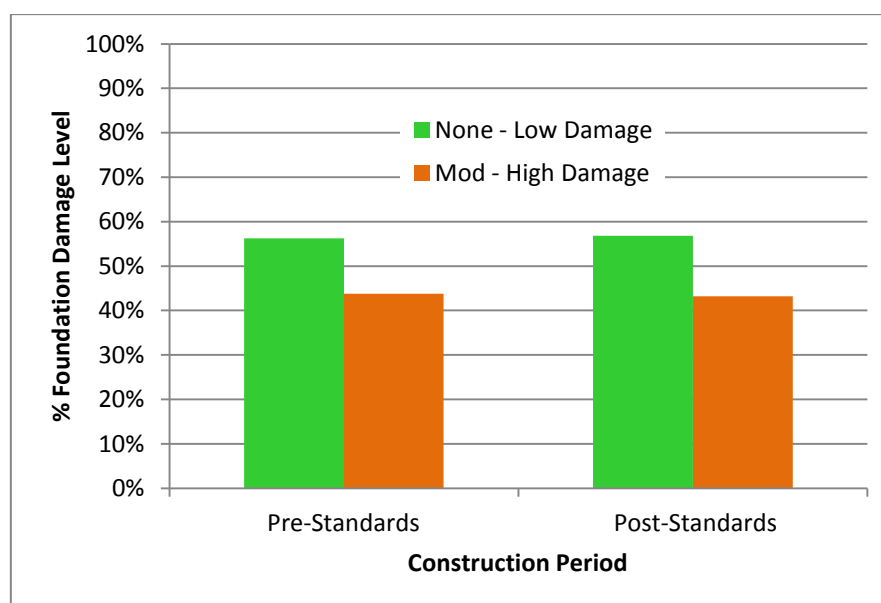


Figure 3.41. Distribution in percentage of concrete perimeter foundation damage for pre and post-standard construction for significant lateral spreading.

Figure 3.42 and Figure 3.43 indicate that houses constructed post-standard suffered more damage to their superstructures regardless of the level of foundation damage. This is likely to be the influence of the increased demand for large open-plan spaces and irregular structural configurations with regard to stiffness and strength in newer houses, as identified by Buchanan et. al. (2011) and Liu and Beattie (2012) and discussed in Chapter 2, Section 2.3.1, but requires more investigation.

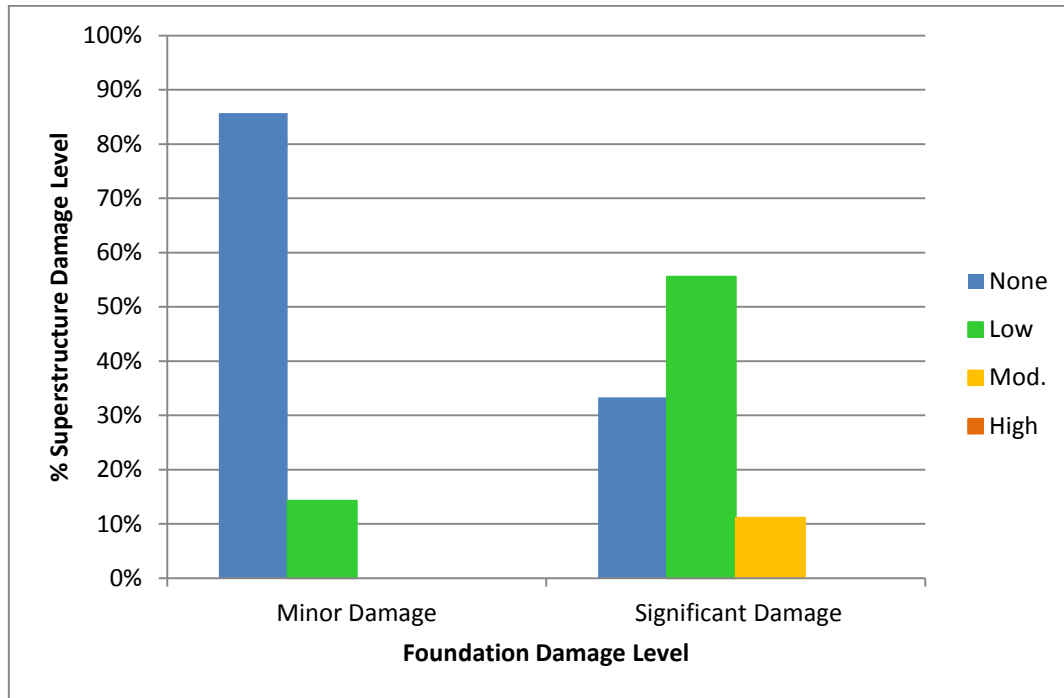


Figure 3.42. Distribution in percentage of superstructure damage for pre-standard concrete perimeter foundations for minor or significant foundation damage.

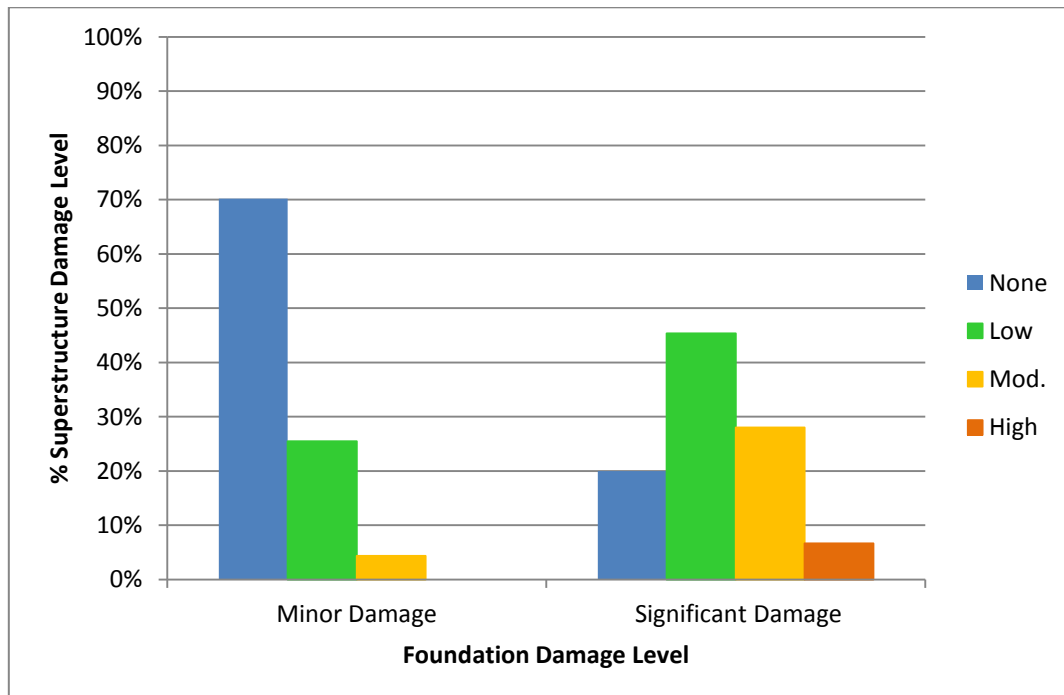


Figure 3.43. Distribution in percentage of superstructure damage for post-standard concrete perimeter foundations for minor or significant foundation damage.

3.6.4.2 Slab-on-grade

The limited data for pre-standard slab-on-grade foundations prevents any definite trends from being identified for foundation damage against inspected liquefaction severity, LiDAR settlement or lateral spreading.

Figure 3.44 and Figure 3.45 indicate that pre-standard slab-on-grade houses are more susceptible to superstructure damage than post-standard houses, regardless of level of foundation damage. However the data on pre-standard houses is limited.

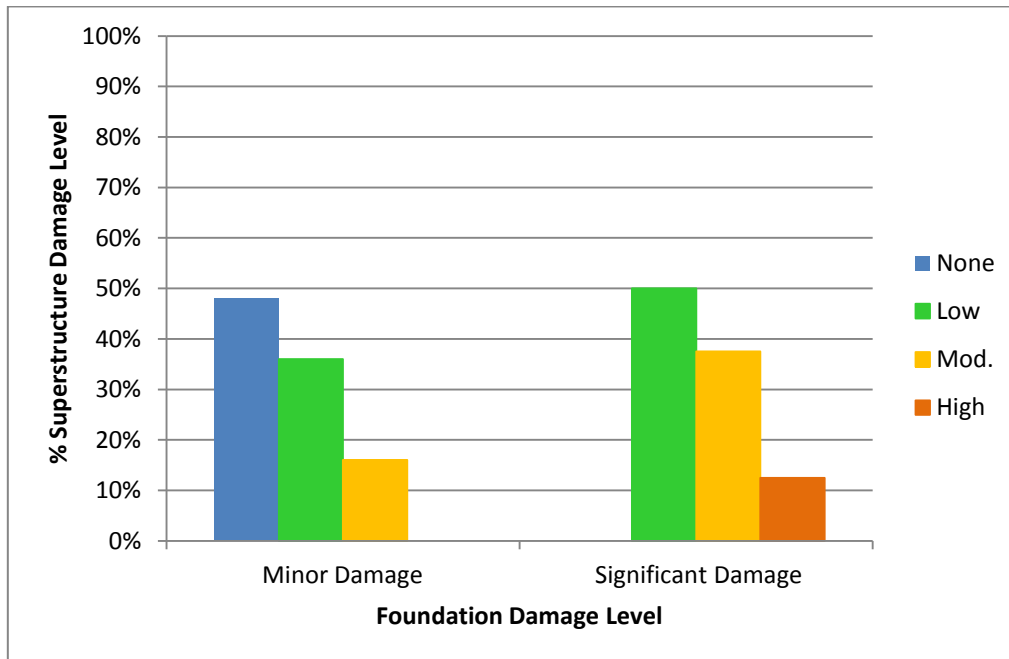


Figure 3.44. Distribution in percentage of superstructure damage for pre-standard slab-on-grade foundations for minor or significant foundation damage.

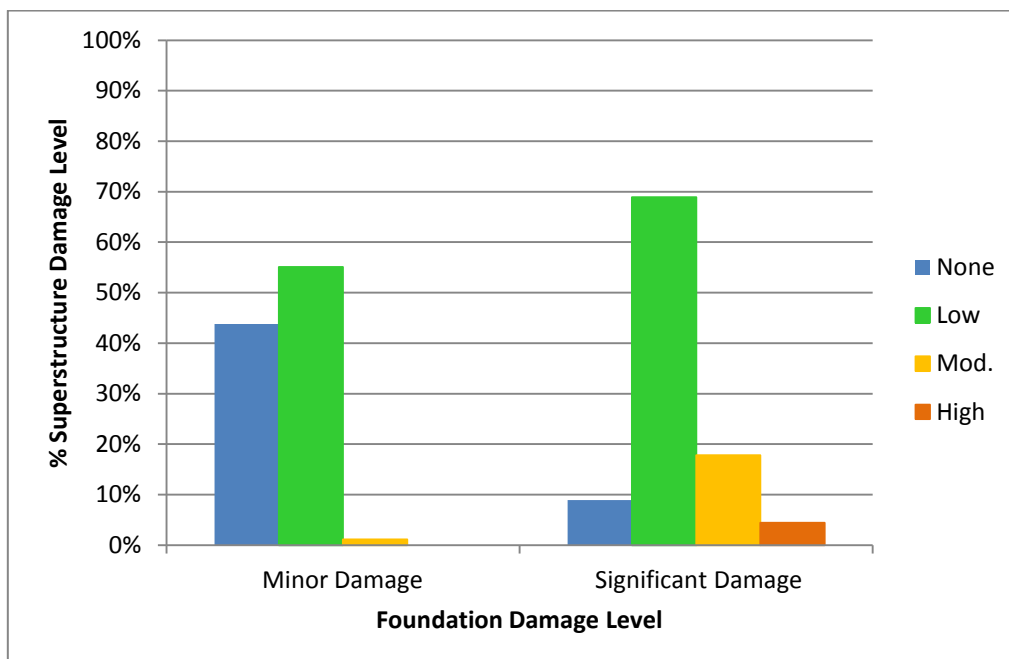


Figure 3.45. Distribution in percentage of superstructure damage for post-standard slab-on-grade foundations for minor or significant foundation damage.

3.6.5 'Light' vs. 'Heavy' Buildings

It was not possible to make any comparisons between the performance of one and two-storey houses with the data collected for houses after the 4th September 2010 earthquake, as there weren't enough two-storey properties surveyed to show any trends. There were only 23 concrete perimeter foundation and 7 slab-on-grade foundation houses inspected that had two storeys, compared to ~270 concrete perimeter and ~170 slab-on-grade one-storey houses.

The data was instead grouped into 'light' and 'heavy' buildings. Heavy buildings were chosen as all two-storey houses, and all houses that had heavy external claddings. Light buildings were all one-storey houses with light external claddings. This still gave greatly skewed data numbers for slab-on-grade houses, as almost all of these had heavy claddings. There were only 12 houses inspected that were both one-storey and had light claddings, and so no conclusions could be drawn on this data. However for concrete perimeter foundation houses this grouping provided quite an even distribution with 163 'heavy' houses and 112 'light' houses. When graphed however, there were no apparent trends in the data for foundation damage against any of the three land-damage indices of liquefaction severity, settlement or lateral spreading. When the superstructure damage was graphed against foundation damage (Figure 3.46 and Figure 3.47), the performance of light and heavy structures was almost identical. This is interesting in itself, as it suggests that light and heavy residential houses performed the same, and there was no disadvantage to having a heavier superstructure. This is likely due to the increased requirements for foundation sections for houses with two storeys and/or heavy claddings in all building standards. This effect is explored further in Chapter 6 where modelling of concrete perimeter foundations is carried out.

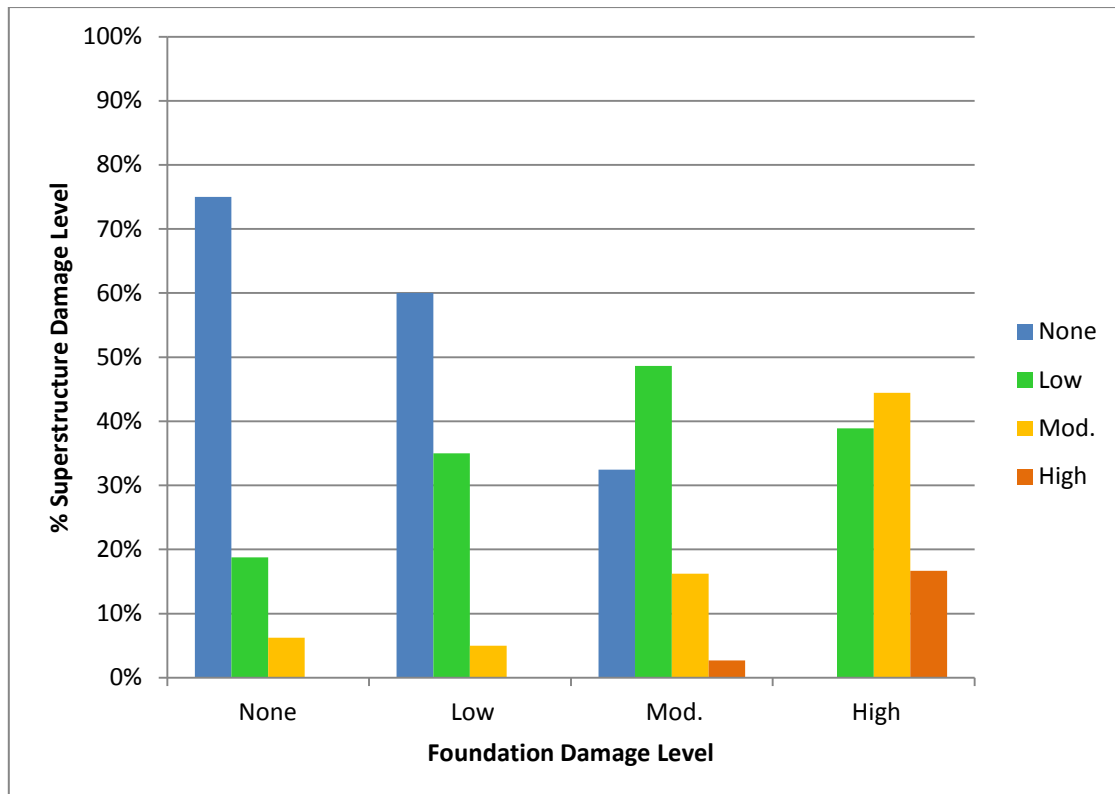


Figure 3.46. Distribution in percentage of damage to house superstructures for differing foundation damage levels for 'light' houses.

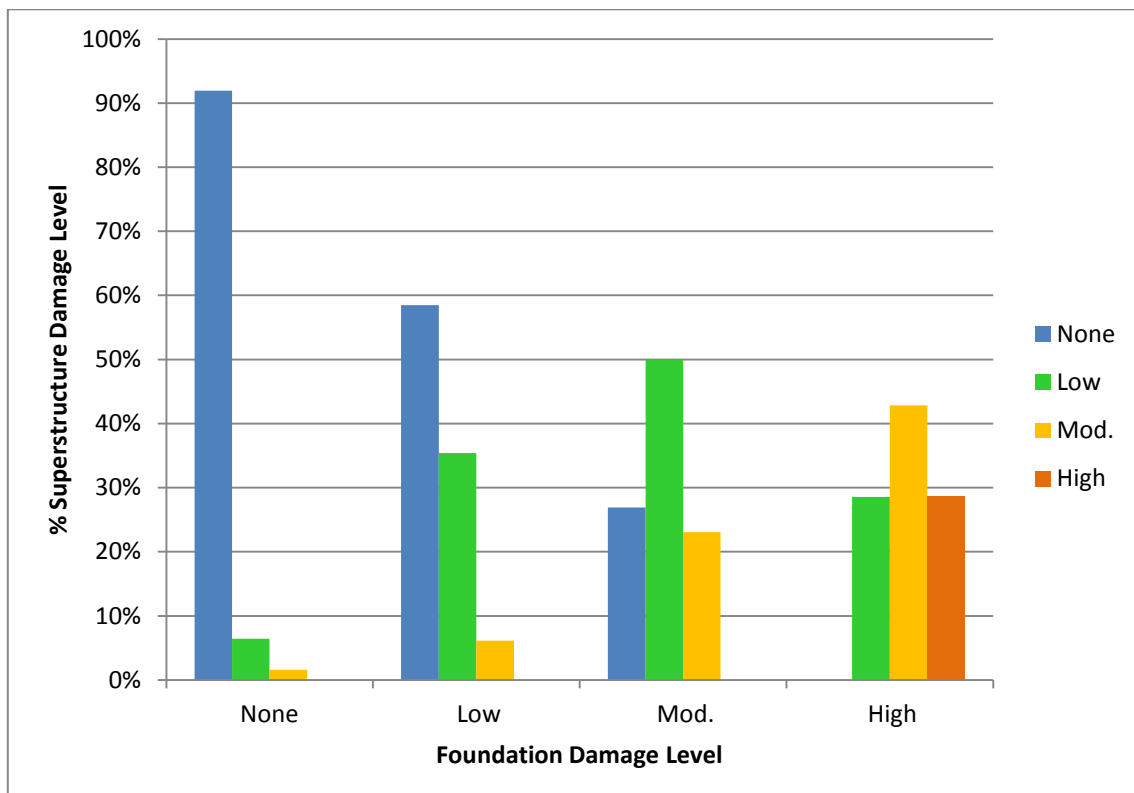


Figure 3.47. Distribution in percentage of damage to house superstructures for differing foundation damage levels for 'heavy' houses.

3.7 Discussion

3.7.1 Foundation Damage

General trends were seen when all the data was combined that for increasing land damage, whether it be inspected liquefaction rating, ground settlement or lateral spreading, the severity of foundation damage also increased. This was expected, due to the large loads and deformation that result when liquefaction and lateral spreading occur.

It is worth noting that although the foundations of houses experiencing a liquefaction rating of ‘none’ had much less damage overall, there was still a number that suffered low damage, with some even suffering moderate damage, as shown in Figure 3.22. This is because the liquefaction rating of ‘none’ does not mean that no liquefaction occurred in the soil beneath a house, but only that there was no visible evidence of liquefaction at the ground surface. The only way to judge the severity of liquefaction at a site without doing extensive ground investigations that take significant time and money, is to assess the level of ground distortion and ejecta seen on the ground surface. The absence of any obvious surface manifestation of liquefaction does not guarantee that there was no liquefaction at depth, but simply that any liquefaction was not severe enough to break through the crust layer to the surface. However, these lower levels of liquefaction can still affect structures such as residential houses located on the ground surface, as the ground still softens, reducing bearing capacity, and moves plastically, which can easily damage the weak foundations used to construct residential houses in New Zealand.

Despite the expected trend observed in the lateral spreading data of increasing foundation damage with the level of lateral spreading, there were still a large number of houses that experienced lateral spreading over 300mm but suffered none or low foundation damage in Figure 3.26. Unless the foundations of these houses are exceptionally strong and stiff (which is unlikely for residential house foundations), further damage would be expected with increased lateral ground movement. A reasonable explanation for these houses with low damage is the nature of the lateral spreading data in the Canterbury Geotechnical Database. The lateral ground movement values are produced from LiDAR data at 4m grid intervals, and averaged to give arrows on the Google Earth map at 56m grid spacings. The averaged nature of this data does not match the physical manifestation of lateral spreading, where the ground movements are localised at cracks that open up in the ground, and are not spread gradually over large distances. These cracks can be large (>200mm wide) or small (<10mm wide) and located close together or spread far apart. This means that residential houses, which generally have footprint dimensions between 10-20m, can easily be situated on a section of land where no cracks are present. With no cracks, they are not directly affected by lateral spreading, where cracks under the footprint will generally cause corresponding cracking and separation in foundations. In this case, foundation damage is more as a result of the underlying liquefaction. The lateral

spreading values from LiDAR data do not take this into account, and so the first assertion that once overall lateral spreading is beyond a certain point foundations are not longer damaged may hold true, as further lateral spreading is going to be manifest in existing cracks widening further, either further damaging the houses that already have cracks under them, or not for houses that don't.

When divided into foundation types, the data suggests that concrete perimeter foundations performed worse than slab-on-grade foundations when subjected to liquefaction. They suffered higher damage to their foundations at high levels of land damage. This is contrary to the findings of Buchanan et. al. (2011) who found that concrete perimeter foundations performed well under liquefaction. However that study was based on rapid non-detailed reconnaissance following the earthquake, so a difference in findings is understandable.

3.7.2 Superstructure Damage

The same general trends were seen for superstructure damage. As the severity of ground damage increases, so does the damage to the superstructure. But at low levels of ground damage there was very little damage to superstructures, with 85% suffering no damage. This supports Buchanan & Newcombe (2010), Buchanan et. al. (2011) and Beattie et. al. (2011) who found that timber-framed houses resisted the inertial loading from the 4th September 2010 earthquake very well. They are very resilient to earthquake loads and for low levels of displacement and foundation damage, there was very little damage to the superstructures. However, they are not flexible enough to resist the permanent displacements and large loads implied at high ground damage levels, resulting in significant structural damage.

The strong trend in superstructure damage when compared to foundation damage (Figure 3.27) indicates that foundation damage is the key parameter that indicates the level of damage that is to be expected in the rest of the structure for residential houses suffering liquefaction effects. Almost all houses with high superstructure damage in Figure 3.27 also suffered high foundation damage. And for these houses with highly damaged foundations there were no houses without at least some superstructure damage. Although the level of liquefaction damage also has a significant influence on the damage to the superstructure, and liquefaction and foundation damage are closely related, as shown in Figure 3.28, if the foundation performs well, it can protect the rest of the house from more severe damage. Conversely, if the foundation performs badly, as for the two houses shown in Figure 3.28 which suffered high superstructure damage at low levels of liquefaction, due to the poor performance of their foundations, superstructure damage is worsened.

3.7.3 Other Trends

It was found that pre-standard concrete perimeter foundations suffered higher levels of damage than post-standard foundations when graphed against liquefaction severity or ground settlement. This is to

be expected, as the pre-standard foundations are a weaker and less ductile construction, generally without any reinforcing. This trend was reversed when the superstructure damage levels of pre and post-standard concrete perimeter foundation houses were compared. This is likely due to the higher demand for open-plan layouts in newer houses, as suggested by Buchanan et. al. (2011) and Liu & Beattie (2012). However, this trend was reversed for slab-on-grade foundations, so needs further investigation.

The data seemed to suggest that the weight of the building did not affect the performance of the house and foundation. This is likely to be the result of the difference in foundation strengths between light and heavy houses, and is further investigated in Chapters 4 and 6. Although the weight of the house may affect any global settlements, these were not measured for enough of the houses inspected to allow the analysis of this parameter.

3.7.4 Limitations of the Data

There were a number of limitations in the data collected for the damaged houses following the 4th September 2010 earthquake. These limited the ability to establish more than simple trends in the data, as more complex statistical analysis was not possible with the data provided.

One of the limitations is that no actual measurements such as floor slope or differential settlement of the structures was recorded consistently. All the damage ratings, such as liquefaction, lateral spreading, foundation and superstructure damage, were recorded on a four-point scale of ‘none’, ‘low’, ‘moderate’ or ‘high’. While these discreet values can be graphed in simplistic ways, they cannot be used for statistical analysis that may have been possible with continuous damage indices.

The spread of the data, with respect to the types of houses inspected and the levels of liquefaction and structural damage suffered to each house prevented some characteristics from being examined. For example, there were not enough two-storey houses inspected to draw any conclusions on the influence of number of storeys on the behaviour of houses, and to some extent the conclusions on pre-standard and post-standard construction were limited in the same way. The lack of houses suffering lower levels of lateral spreading also meant that the effects of lateral spreading couldn't really be examined. The purpose of the data collection on houses after the September earthquake used in this chapter was not to collect an evenly-distributed data set for the purposes of the analysis of all these different factors, but rather to inspect a large number of houses that suffered significant land damage as a result of liquefaction so as to allow the determination of the effect of liquefaction on residential houses. This aim was achieved, and in addition a number of other factors influencing the behaviour were able to be examined in general terms, making this a valuable data set.

The other limitation is the fact that the assessments were conducted by a number of different teams. As the assessment criteria for the different damage indices are somewhat subjective, this leads to the

possibility for discrepancies in the data due to the various inspectors interpretations of ‘low’ or ‘high’ damage for each index. These possible discrepancies were somewhat reduced by the post-assessment collation of the data and re-examination of each damage index by the same two people within the summer research team. During this process, inspection sheets, recorded notes and photographs were all examined to determine if the damage level recorded during the inspection for each index matched to the damage ratings outlined in Section 3.3.2.2. The data on land damage from the Canterbury Geotechnical Database was also included to provide external, non-biased values for different land-damage indices. The use of the vertical and horizontal ground movements derived from LiDAR data served to provide alternative land damage values that were unaffected by human error. As many of the trends seen in the data graphed with the liquefaction damage index recorded in these inspections were matched by the same graphs with the LiDAR ground settlement data, the land damage levels recorded by the summer research team can be relied upon, and the trends shown are due to the behaviour of the houses inspected.

The various trends found in the data for foundation damage and superstructure damage, along with the performance of concrete perimeter and slab-on-grade foundations, were actually all found to be stronger when graphed against the inspected liquefaction severity rating from the summer research team, than when graphed with the LiDAR data, either settlement or lateral spreading. This tendency can be seen between Figure 3.22, Figure 3.24 and Figure 3.26 in Section 3.6.1, and in Section 3.6.3 for the comparisons between the two foundation types. This suggests that the data collected by the summer research team is a very valuable data set that has shown strong trends in the performance of residential houses subjected to liquefaction. Although the data collected by the summer research team was somewhat subjective and simplified in nature, the fact that it was localised and specific to each house inspected has resulted in data that is more accurate than the LiDAR data that was measured from unrelated maps. For the trends in lateral ground movement against building and foundation damage this is most likely due to the nature of the lateral spreading data for the Canterbury Geotechnical Database. The lateral ground movement values are produced from LiDAR data at 4m grid intervals, and averaged to give arrows on the Google Earth map at 56m grid spacings. The averaged nature of this data does not match the physical manifestation of lateral spreading, where the ground movements are localised at cracks that open up in the ground, and are not spread gradually over large distances. These cracks can be large (>200mm wide) or small (<10mm wide) and located close together or spread far apart. This means that residential houses, which generally have footprint dimensions between 10-20m, can easily be situated on a section of land where no cracks are present. With no cracks, they are not directly affected by lateral spreading, where cracks under the footprint will generally result in corresponding cracking and separation in foundations. This means that the foundation damage is more heavily influenced by the underlying liquefaction. The lateral spreading values from LiDAR data do not take this into account, resulting in weaker trends.

Despite the limitations discussed, the data collected by the summer research team still provides a valuable insight into the performance of houses that were affected by high levels of liquefaction and subsequent land damage in the 4th September 2010 earthquake. This is confirmed by the comparison in trends between the collected land damage data and the LiDAR data sets.

3.7.5 Summary of Key Findings

There were a number of trends discovered in the performance of residential houses that suffered significant liquefaction and lateral spreading damage in the 4th September 2010 earthquake. These trends are summarised here:

- For all houses there were general trends of an increase in the proportion of moderate and highly damaged foundations as the severity of ground damage increased.
- The proportion of moderate and high superstructure damage to all houses increased with higher levels of foundation damage and liquefaction.
- At high levels of land damage, for liquefaction, settlement and lateral spreading, concrete perimeter foundations had a higher proportion of moderate and highly damaged foundations than slab-on-grade, suggesting slab-on-grade foundations perform better when subjected to liquefaction effects.
- At high levels of foundation damage, houses with concrete perimeter foundations suffered more superstructure damage than houses with slab-on-grade foundations.
- Pre-standard concrete-perimeter foundations suffered higher levels of damage than post-standard foundations when compared to liquefaction and settlement, which is to be expected as pre-standard foundations are weaker and less ductile, due to their poorer construction. However the performance of the two different age-period foundations under lateral spreading was almost identical. This was assumed to be due to the poor lateral resistance of the concrete perimeter design, but needs to be investigated further.
- Buildings with concrete perimeter foundations appeared to suffer higher superstructure damage in post-standard houses, which could be due to higher demand for open-plan layouts in newer houses. However this trend was the opposite for slab-on-grade houses, with old houses suffering more superstructure damage.
- There was insufficient data to draw any conclusions between the performance of one and two-storey houses, and no trends were found for 'light' or 'heavy' perimeter houses, suggesting the weight of the superstructure for perimeter houses does not affect performance.

4 The Performance of Foundations Observed from Detailed Inspections after the 22nd February 2011 Christchurch Earthquake

4.1 Introduction

The M_w 6.2, 22nd February 2011 Christchurch earthquake produced far more severe and extensive liquefaction than the 4th September 2010 Darfield earthquake, with larger peak ground accelerations of 0.2-0.6g experienced by the majority of Christchurch city (O'Rourke et. al., 2012). Liquefaction was widespread across the eastern suburbs of Christchurch, and along the Avon River, with moderate liquefaction stretching across the suburbs north of the Central Business District.

In Chapter 3 it was identified that foundations play a key role in determining the damage to house superstructures under liquefaction loads and that different types of foundations may behave differently under the effects of liquefaction. To investigate this, it was decided to conduct systematic inspections of houses with four different foundation types. These were: Concrete Perimeter with short piers (constructed to NZS3604 and variations), Concrete Slab-on-grade (constructed to NZS3604), RibRaftTM slab foundations (Firth Industries 2003) and deep piled foundations. Around 40 houses of each foundation type were inspected across a wide range of liquefaction severities, to better categorise the performance of each foundation type. An additional 19 inspections were conducted of houses that had not experienced any liquefaction to provide a benchmark level for comparison.

The results of the inspections were analysed and a number of trends identified. This chapter outlines the methodology used to select the houses for inspection and how the inspections were conducted. It also outlines typical damage modes observed for the four foundation types, presents the results of the data analysis and discusses the trends observed.

4.2 The Foundation Types

The four foundation types of Concrete Perimeter, Slab-on-grade, RibRaftTM and deep piles, were chosen as they are the four most common types of foundation used for residential houses in Christchurch and in particular on the flat where liquefaction occurred. They also represent the large majority of house foundations throughout New Zealand, which will allow the results of this study to be used nationwide.

4.2.1 Concrete Perimeter

Most older houses constructed before 1980 in Christchurch and New Zealand have concrete perimeter foundations with short piers, shown in Figure 4.3. The primary vertical and lateral load-resisting component of this foundation type is a concrete perimeter wall, which is generally 150-300mm wide, cast into the ground 100-300mm and extending above the ground 200-600mm. This supports most of the weight of the roof, the external walls and tributary areas of the interior walls and can be constructed of bricks or various grades of concrete which can be unreinforced or reinforced in different layouts, depending on the age of construction and corresponding housing standard. The biggest change in construction methods came with the introduction of N.Z.S.S. 95 in 1935. Prior to 1935, there was no standard of construction for house foundations. The concrete perimeters were made of concrete with widely varying properties, were often unreinforced, and used large stones, bricks or other hardfill to reduce the volume of concrete required (see Figure 4.1). With the introduction of standards-based construction, the dimensions, reinforcing content and layout and concrete strength of the perimeter foundation walls all became prescribed. This ensured a better quality of construction. A review of housing standards can be found in Chapter 2, Section 2.5.1, with summary drawings for the foundations of each period of construction in Appendix A.



Figure 4.1. Concrete perimeter wall cross-section, showing large stones used as fill.



Figure 4.2. Round tapered pier supports, showing connection to bearers.

The secondary vertical load-resisting component of this foundation type are the short pier supports that support the interior floor (Figure 4.2). These are entirely separate from the perimeter wall, and only support the weight of the floor and some internal walls. They can be timber, pre-cast or cast-in-situ concrete with a range of reinforcing. They are generally cast into the ground by 100-300mm, and extend anywhere from 100-600mm above the ground to support the floor bearers. They are connected

to the floor above with wire hooks, which do not provide a rigid connection, but simply hold them in place. It is not uncommon, particularly in older houses, to find that this attachment has been neglected, and the floor is just sitting on top of the piers.

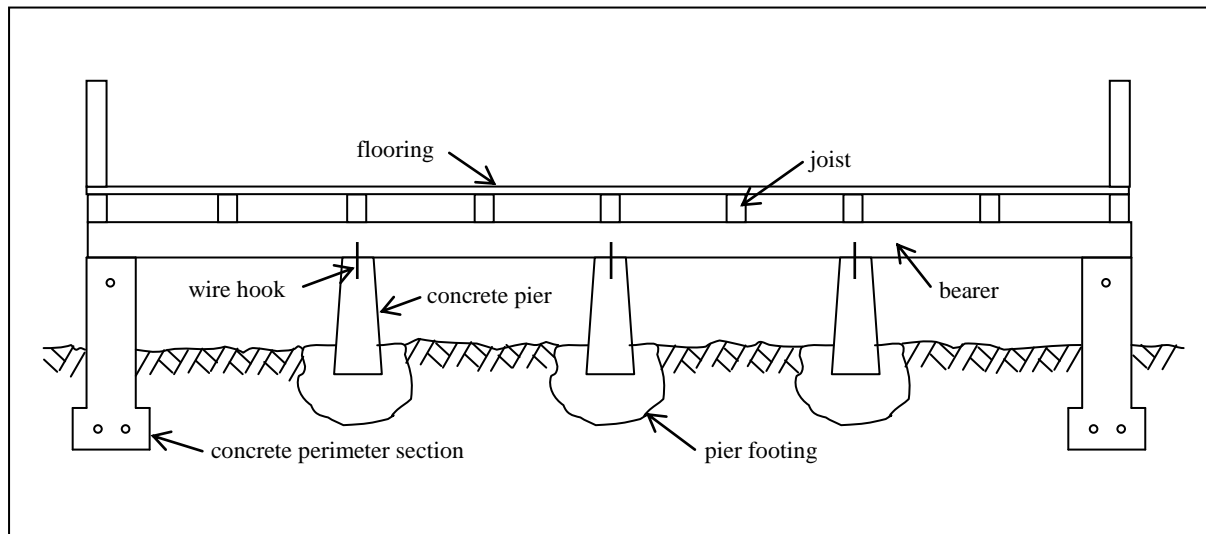


Figure 4.3. Generalised cross-section of a concrete perimeter foundation and floor structure to NZS3604.

From here on this foundation type is referred to as concrete perimeter. For more information, see the standards N.Z.S.S. 95, NZS1900 and NZS3604.

4.2.2 Slab-on-Grade

Concrete slab-on-grade is the most popular foundation for standard newer houses constructed after 1980 in Christchurch and New Zealand. The standard design, shown in Figure 4.4, consists of a reinforced concrete ring foundation wall which is generally between 150-300mm wide and extends into the ground 300-600mm. Inside this under the footprint of the house is hardfill (Figure 4.5). This layer can be anywhere from 75-600mm thick, and in Christchurch mostly consists of river-run tailings; round, uniformly graded gravel between 20-60mm in diameter. This ranges from being uncompacted, to being compacted in 150mm layers “until the material is tightly bound together and does not visibly deform under the weight of a pressed adult heel” (NZS3604:1999). The slab is then cast over the top of this hardfill, with a damp-proof membrane in-between. The slab is generally ~100mm thick and reinforced with either 665 or 663 wire mesh, which is a grid of, respectively, either 5.3mm or 6.3mm diameter wire at 150mm centres in both directions (Fletcher Reinforcing, 2013). In some cases there is additional reinforcing around the edges of the footprint, or under internal load-bearing walls where a thickening of the slab is generally required.

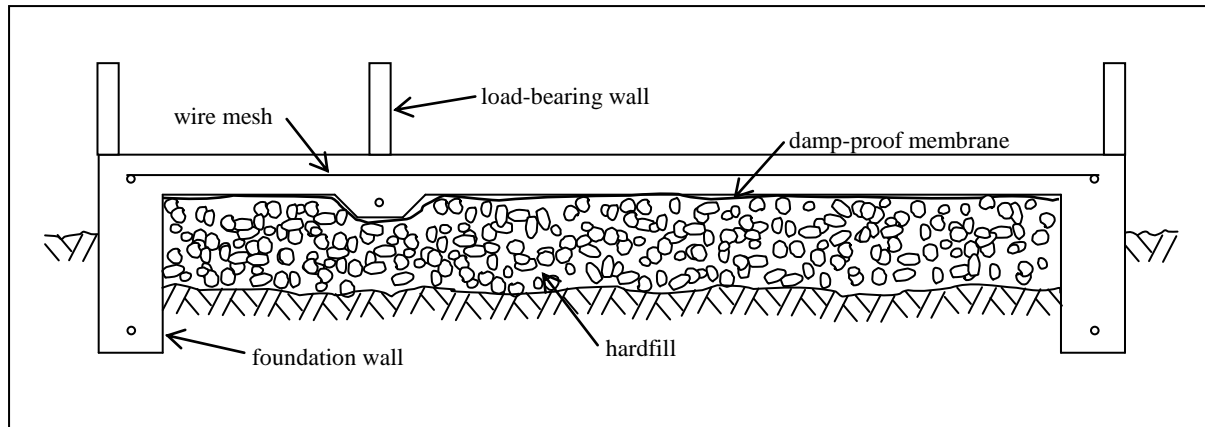


Figure 4.4. General cross-section of a slab-on-grade foundation and floor structure to NZS3604.

The superstructure of the house rests directly on the slab. From here on this foundation type is referred to as slab-on-grade. For more information see Appendix A and NZS3604.



Figure 4.5. Exposed reinforced concrete ring foundation wall under a slab-on-grade foundation, with river-run hardfill inside.

4.2.3 RibRaft™

As discussed in Chapter 2, Section 2.5.1.6, RibRaft™ foundations are a proprietary flooring system designed and distributed by Firth Industries. They were chosen for inspections as prior to the earthquakes they were becoming a popular foundation choice for large and more complicated residential buildings. Since the earthquakes, the guidance on repairing and rebuilding foundations released by the Ministry of Business, Innovation and Employment has recommended a number of new foundation solutions for TC3 properties that are very similar to, or variations of, the RibRaft slab. For further information on this see Chapter 2, Section 2.5.3, and MBIE (2012).

As shown in Figure 4.6, RibRaft slabs are built on top of the ground, unlike standard NZS3604 slab-on-grade foundations which have the ring foundation wall. This means (theoretically) that they can move separately from the ground in the event of an earthquake, as they are isolated by a layer of compacted sand that is put down prior to construction between the foundation and ground below. This prevents the foundations from being pulled with the ground as it moves laterally during liquefaction and lateral spreading, reducing damage. Their structure consists of a grid of reinforced concrete

beams, separated by polystyrene pods, with an 85mm thick concrete slab on top, which is reinforced with wire mesh. The whole foundation is prepared with foam pods and reinforcing (as in Figure 4.7) and then cast in one as a monolithic unit. From here on this foundation type is referred to as ribraft.

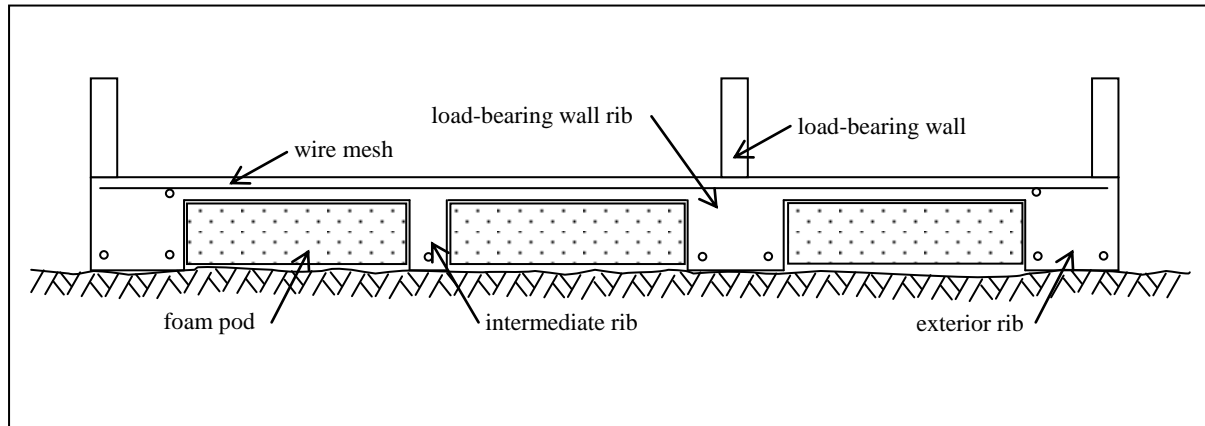


Figure 4.6. General cross-section of a ribraft foundation and floor system (Firth, 2003).



Figure 4.7. RibRaft slab prior to concrete pouring, showing foam pods and reinforcing steel.

4.2.4 Deep Piles

Prior to the earthquakes, deep piles were generally only used on the flat for houses where underlying soft soil conditions such as peat were expected to be a problem for static long-term settlement, and also for larger two and three storey town house type units, as the effects of liquefaction were not previously considered in housing design or construction. Because of this there are only a small number of these foundation types in Christchurch, and they are generally limited to specific areas. As for the RibRaft foundations, following the earthquakes, piled foundations are being considered as solutions to requirements for better foundations in TC3 areas (MBIE, 2012).

The piles used are either 150mm diameter, H5 treated timber, or the more common concrete piles which can range from 75x90mm to 150x150mm and are reinforced with four wire strands running the length of the pile. The piles are driven into the ground until a suitable bearing resistance is met. This is determined by setting a maximum distance for the piled to travel in one drop of the hammer, which depends on the pile capacity required, and the hammer weight and drop height. This means that the

depth of piles is not always consistent across a site, often with a depth range of 0.5-1.5m within the building footprint. The final driven pile depth commonly ranges from 3.5m to 7.5m, but can be as shallow as 2m or as deep as 9m. The piles are then cut off at the same level, about 200-400mm above the excavated ground level (Figure 4.8.(a) and Figure 4.9.(a)). Pile caps are then cast around the exposed pile tops to protect them, and ring foundation walls cast over the edge piles, to allow filling of the space with hardfill (Figure 4.8.(b) and Figure 4.9.(b)). The floor slab is then cast on top of all this as for a standard slab-on-grade foundation, or a timber floor is constructed as for concrete perimeter foundations. From here on this foundation type is referred to as piled.



Figure 4.8.a) Driven concrete piles cut off above ground level ready for pile caps. b) Pile caps cast over the same piles.



Figure 4.9.a) Driven concrete piles cut off in a trench excavated for a ring foundation wall. b) Ring foundation wall cast over the top of piles.

4.3 Methodology

4.3.1 Selection of Houses

The aim of the study was to inspect at least 40 houses of each foundation type across areas which had suffered a range of liquefaction severities in the 22nd February 2011 earthquake. This number was decided based on the aim of inspecting approximately 10 houses of each foundation type in each of

four broad land damage categories to provide a data set with adequate coverage and depth to draw relevant conclusions on the behaviour of the different foundations across all levels of liquefaction. The intended spread of these inspections is shown in Table 4.1.

Table 4.1. Number of houses intended for inspection in the different foundation and land-damage categories.

Foundation Type	Land Damage Category				Total
	Extreme	High - Moderate	Moderate - Low	Low-None	
Concrete Perimeter	10	10	10	10	40
Slab-on-Grade	10	10	10	10	40
Ribraft Slab	10	10	10	10	40
Piled	10	10	10	10	40

To conduct the inspections, areas of interest for the different levels of land damage had to be identified. This was done using the University of Canterbury Liquefaction Map, shown in Figure 4.10 (reproduced from Cubrinovski & Taylor, 2011). Because there were limited numbers of ribraft and piled foundations within the liquefied areas shown in Figure 4.10, lists of locations of houses with these foundation types in Christchurch were obtained first, supplied by Firth Industries and Boulton Piling for ribraft and piled foundations respectively. Houses from these lists that fell within the identified liquefied areas in Figure 4.10 were then identified and marked for inspection. These houses were visited first, and if they could be inspected, a concrete perimeter and a slab-on-grade foundation house were inspected close by.

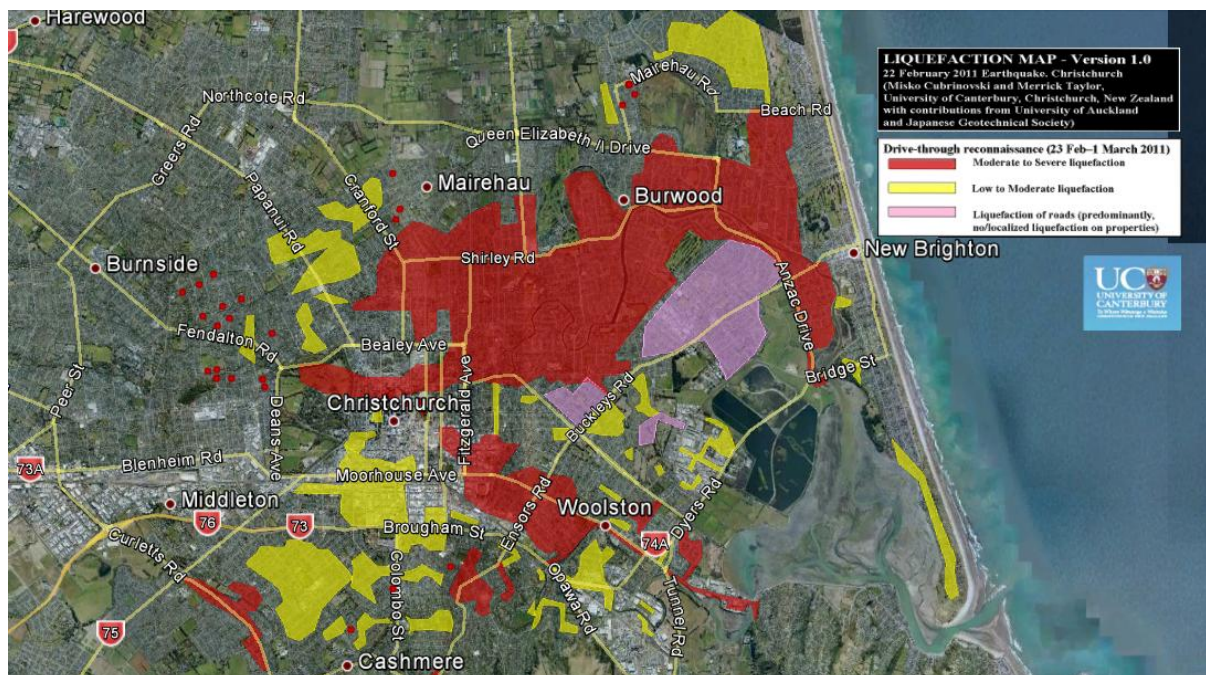


Figure 4.10. Areas of liquefaction (in Christchurch) in the 22nd February 2011 earthquake (Cubrinovski & Taylor, 2011).

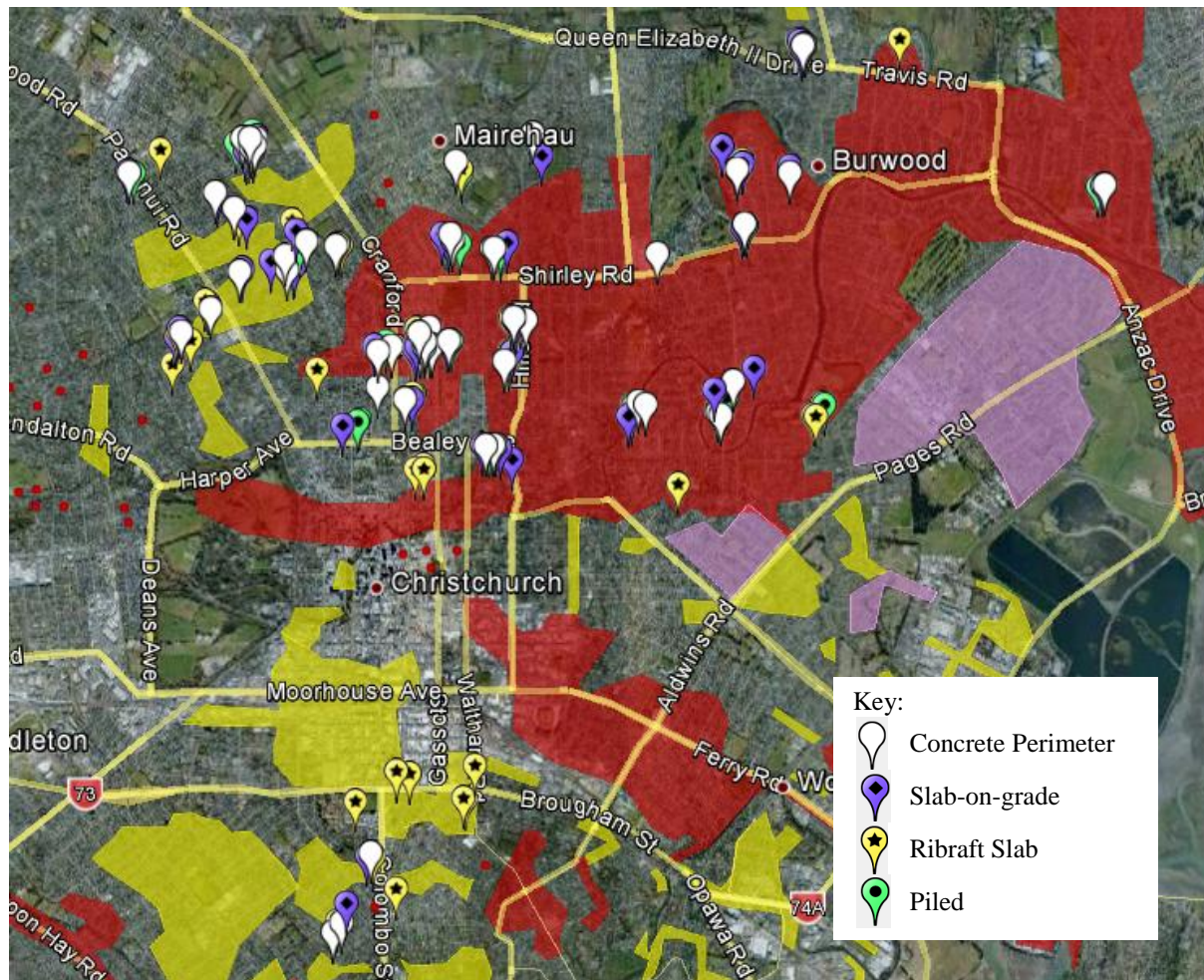


Figure 4.11. House inspection locations overlaid on the UC Liquefaction Map, showing different foundation types.

4.3.1.1 Local Proximity of Inspections

The proximity of inspections of each house foundation type was very important. Inspecting houses with the different foundation types close to each other meant houses that suffered very similar levels of liquefaction and ground damage and the same intensity of ground motion could be comparatively examined. This allowed for more direct and reliable correlation between the liquefaction and land damage parameters, and comparison between the foundation types.

This was achieved by first inspecting a ribraft or piled foundation house, and then inspecting a concrete perimeter and a slab-on-grade house close by, preferably within 100m. Examples of the proximity of inspected houses are shown in Figure 4.12.

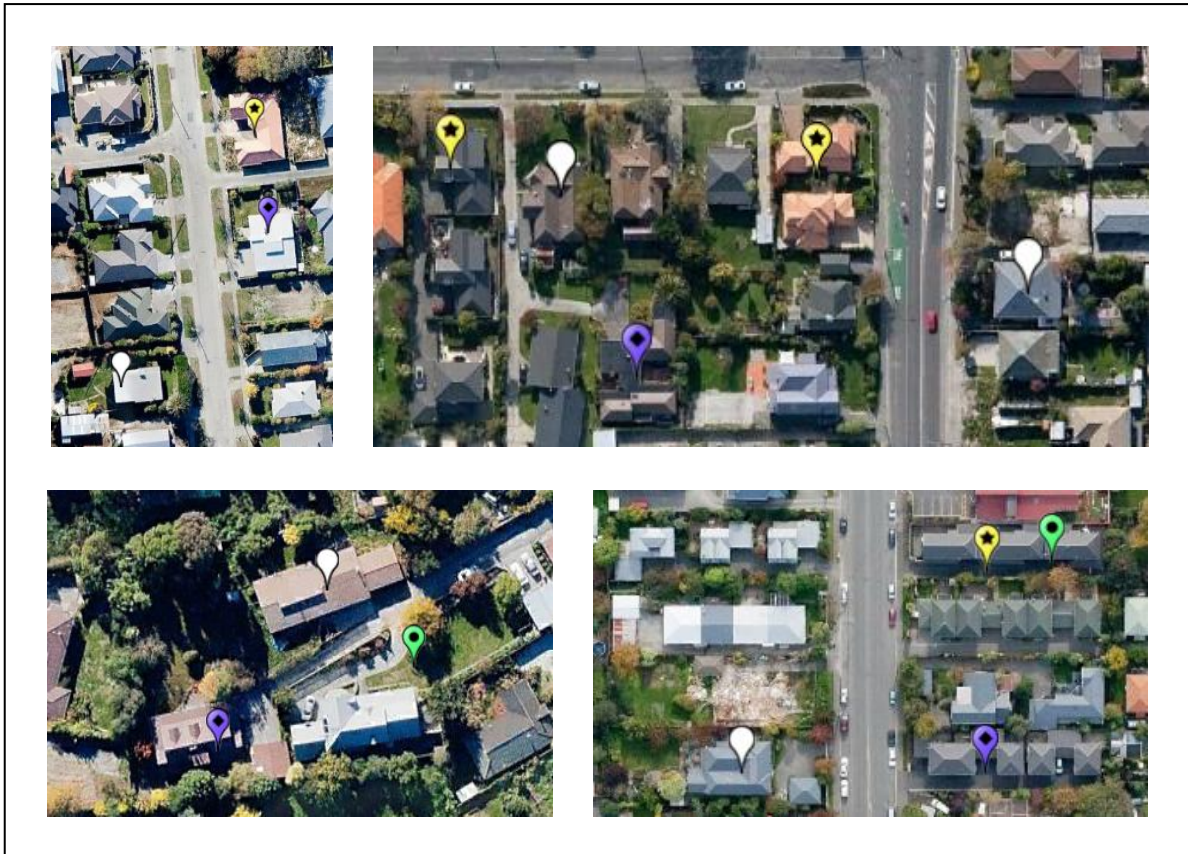


Figure 4.12. Examples of the proximity of different foundation types inspected.

4.3.1.2 Difficulties in Obtaining the Intended Data

Figure 4.11 shows the locations of all houses surveyed for this study, with the total number of inspected houses for each foundation type shown in Table 4.2. Overall, the intended coverage of inspections was achieved, with around 40 houses of each foundation type inspected, across a range of liquefaction severities. However these were not divided as evenly as was indicated in Table 4.1, due to a number of factors.

During the inspections, each property was given an individual rating for its liquefaction and lateral spreading damage, rather than using overall values for each suburb or area from the UC Liquefaction Map that were first used to identify the areas of interest for inspection. This was done because of the significant local variation in liquefaction severity and effects which can cause large differences in the loads felt by different structures within the same ‘moderate’ or ‘severe’ liquefaction areas in Figure 4.10. This means that it was difficult to get an even distribution of inspected houses across the range of liquefaction severities, as prior to the inspection, the actual liquefaction severity at a single property was not known.

The low number of ribraft and piled foundations combined with failure to obtain permission from the owners to conduct inspections at some houses meant that the spread of these houses over the range of land damage severities was not ideal. All available houses with ribraft and piled foundations had to be

inspected, but they could not be chosen to achieve a perfect spread of land damage severities. This affected the spread of concrete perimeter and slab-on-grade houses, as they were inspected close to ribraft or piled houses.

There were more concrete perimeter and slab-on-grade foundation houses inspected, as these were inspected close to ribraft or piled foundation houses, which were not always close enough together to be in pairs.

Table 4.2. Number of houses of each foundation type actually inspected.

Foundation Type	# of houses inspected
Concrete Perimeter	49
Slab-on-grade	43
RibRaft Slab	39
Piled	39

A range of house types were accepted for inspections, with variations in the age of construction, type of cladding on the exterior walls and roof, layout shape and size and number of storeys. This was due to the lack of numbers of ribraft and piled foundation houses, preventing the ability to choose more specifically. While a more specific data set, say all one-storey houses with a particular cladding and roofing material in a narrow age range would most likely provide more robust data for the comparison between foundation performance, the data set that was collected allows further analysis of a number of the house construction details that were identified as having a possible influence on performance that were identified in Chapter 3.

4.3.1.3 No-Liquefaction Inspections

Ten slab-on-grade and nine concrete perimeter foundation houses from the western suburbs of Christchurch were also inspected. This location, shown in Figure 4.13, was chosen as there was no liquefaction there, and it experienced a similar level of inertial shaking to the rest of the inspections. If there was any soil softening, it was insignificant when compared to that suffered by the houses in the main body of inspections.

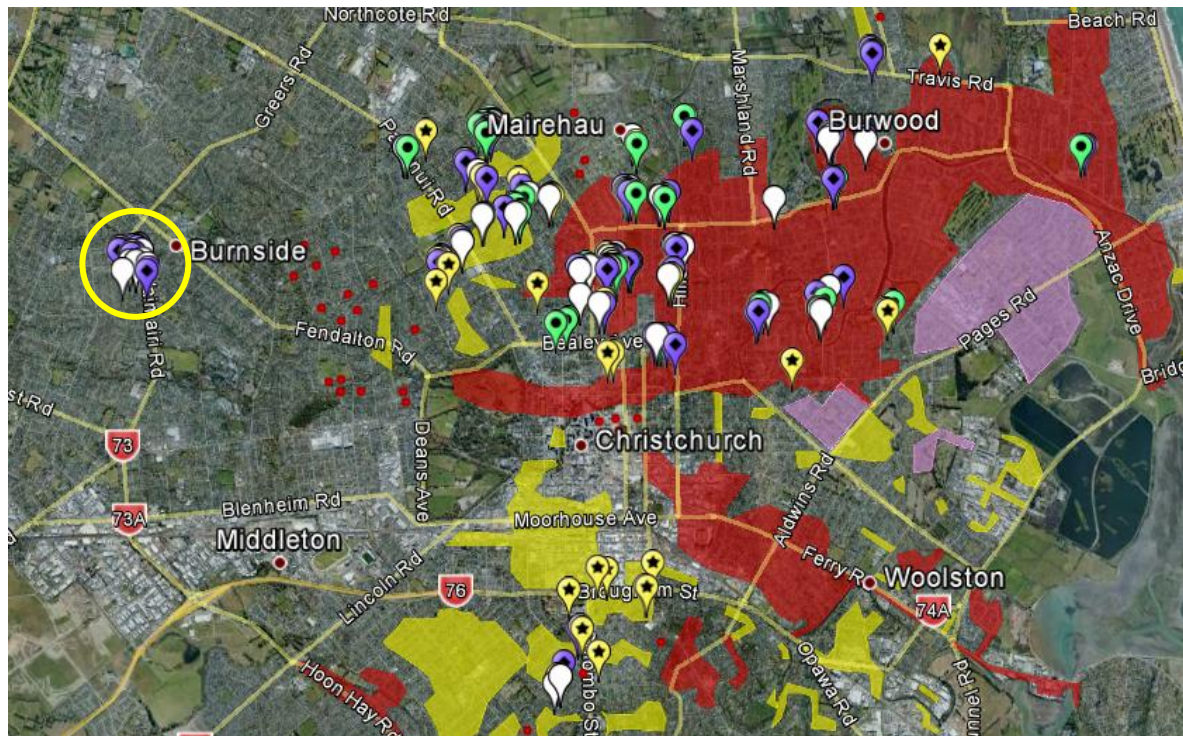


Figure 4.13. No-Liquefaction inspections, highlighted inside the yellow circle in the West of Christchurch, shown in relation to the location of the other inspections.





These no liquefaction inspections provide a baseline level of damage, representative of the level of damage that was suffered in the Canterbury earthquake sequence, by houses that were not subject to the effects of liquefaction, and allow the results of the other inspections to be put into context. Both foundation types were inspected as they deform quite differently and it is important to have a comparison for both. But these were the only two needed as ribraft and piled foundations behaved very similarly to slab-on-grade foundations at low levels of damage with no liquefaction.

Although some of the houses inspected in the range of liquefaction zones had no surface manifestation of liquefaction around them (and were given liquefaction damage ratings of ‘none’), this does not mean that there was no liquefaction at depth, or significant softening of the ground as it approached the threshold for liquefaction. This can still cause significant damage to the foundation and superstructure. Concrete perimeter and slab-on-grade foundations with no liquefaction both had to be inspected, as the two types of foundations deform quite differently.

4.3.2 Equipment

The inspections conducted by the author for this research were more detailed than those done following the 4th September 2010 earthquake by the summer research team in Chapter 3. In addition to the subjective ratings that were used in the earlier inspections, a number of measurements of floor slopes and levels were also taken in these inspections to provide quantitative data that could be used to represent the level of foundation damage for graphing and analysis. Table 4.3 shows the equipment that was used during all inspections, to gather the range of data required.

Table 4.3. Equipment used during house inspections.

Equipment/ survey tool	Photo	Use	Units and Accuracy
Ruler/tape measure		<ul style="list-style-type: none"> - Measuring crack widths - Measuring distances for estimation of total settlements and lateral spreading 	<ul style="list-style-type: none"> - Millimeters - +/- 1mm
Smart Level		<ul style="list-style-type: none"> - Measuring tilt angles of walls and local floor slopes 	<ul style="list-style-type: none"> - Degrees - +/- 0.1°
Zip Level		<ul style="list-style-type: none"> - Measuring the relative differential settlement and distortion of the floor between points 	<ul style="list-style-type: none"> - Millimeters - +/- 2mm
Camera		<ul style="list-style-type: none"> - Documenting damage (cracks, tilt etc.) 	<ul style="list-style-type: none"> - N/A

4.3.3 Inspection Methodology

In early 2011, during the development stage for this research project, around 30 preliminary inspections were carried out on a range of properties and foundation types across the spread of liquefaction severities. The purpose of this was to see how the inspections would work and identify the key parameters in measuring the damage of each building. These inspections used an inspection form (shown in Appendix F) that was a more detailed version of the inspection form used by the summer research team following the 4th September 2010 earthquake (shown in Appendix D). The following discussion is about the final inspection process that was developed after conducting these preliminary inspections.

The inspection process was non-invasive. No under-floor or roof spaces were entered during the inspections and only visible damage to the interior and exterior of the house and the surrounding

property was recorded. Upon arriving at a house that was intended for inspection, the owner was spoken to and permission to conduct the inspection was obtained. An information letter (Appendix J) was provided if wanted. If the owner was not there, and the house was of interest for inspection, then a permission form (Appendix H), the information letter and a questionnaire (Appendix I) were left in the letterbox. Originally, the permission letter was used as a way of communicating with the owner indirectly to either arrange a meeting time to conduct the inspection, or obtain permission in writing to conduct an external inspection while the owner was absent. In the case of permission being granted for an external inspection, the questionnaire was provided to gain answers from the owner on the performance of their house that would otherwise be discussed in person. However, it was decided after the preliminary inspections were conducted that it was necessary to enter each house to record floor levels and so the permission form was then only used as a way to communicate with the owner to arrange a time to come back.

Once permission was obtained, the inspection of each property followed the same overall process, which included the following steps (refer to the example inspection sheet in Appendix G):

- Recording physical details of the house and surroundings (address, ground slope and distance to water of a free face if applicable, age, number of storeys, floor plan size and shape, types of foundation, floor, framing, cladding and roof).
- Recording the land damage (both liquefaction and lateral spreading) suffered by the property in each major seismic event. This was done through discussion with the owner and any signs that were observable at the time. See Table 4.4 for the ratings used.
- Walking around the property to record the level and extent of damage to any surrounding structures such as decks, steps, paving, fences and walls.
- Visual inspection and recording of the level and extent of damage to external and internal cladding, windows, doors and the roof.
- Sketching of the floor plan (ground floor only for 2-storey) on which is recorded locations and sizes of any cracks in the foundation.
- Taking tilt angles with the Digital Spirit Level at multiple points throughout the house, in two directions at each point, with an average of at least four readings per room. Even if the house is totally un-damaged, these levels are still taken. These are also recorded on the floor plan sketch.
- Taking floor levels with the Zip Level throughout the house, with at least one point close to each corner of each room, and at least one in the middle of each room. These levels are also recorded on the sketched floor plan.

4.3.4 Data Processing

4.3.4.1 Data Recorded

The inspection sheet that was used for the main inspections (Appendix G) was expanded on from the original inspection sheet (Appendix F), using some of the features of an inspection sheet used by BRANZ inspectors in separate house inspections conducted around the same time for other research, such as Liu & Beattie (2012), Thomas & Shelton (2012) and Thomas et. al. (2013). The BRANZ inspection sheet mainly utilised percentage bracket ratings to describe the level of damage to various structural components. For example, in a lightly damaged house, '10-20%' of the internal lining suffered cracking at the corners of openings such as door and window frames. In a badly damaged house this might be 50-75%. This method of measuring damage was brought into the inspection sheet for this research to allow some cross-over of the data collected in the parallel studies. It was used in combination with the discreet ratings used by the summer research team following the 4th September 2010 earthquake, which were extended to include a top rating of 'severe' for a total of five damage categories. The following data was recorded for each inspection on the inspection sheet:

General Information

- Property address
- Owner's name and contact details
- Date
- Examiner
- EQC damage sticker
- Flat or sloping site
- Distance to water if appropriate
- Age band, Defined by standards periods (Chapter 1, Section 5.1)

Structural Information

- Number of storeys
- Building size (footprint)
- Plan shape Rectangle, "T", "L", Complex
- Foundation type
- Floor type
- Floor height
- Framing material
- Cladding material
- Roof material

Ground damage

- Liquefaction severity in each major event 5-point scale, 'none' to 'severe'
- Lateral spreading severity in each major event 5-point scale, 'none' to 'severe'

General building deformation

- Stretching 'low', 'moderate' or 'high'
- Hogging 'low', 'moderate' or 'high'
- Dishing 'low', 'moderate' or 'high'
- Racking/Twisting 'low', 'moderate' or 'high'
- Tilting 'low', 'moderate' or 'high'
- Differential Settlement 'low', 'moderate' or 'high'
- Differential Displacement 'low', 'moderate' or 'high'

Damage to surroundings

- Decks 5-point scale, 'none' to 'severe'
 - o Settlement
 - o Tilting
 - o Cracking
 - o Separation
- Concrete Paving 6-point percentage scale:
 - o Settlement 'none', <10%, 10-20%, 20-50%, 50-75%, 75-100%
 - o Tilting
 - o Cracking
 - o Separation
- Steps Recorded value of each, if applicable
 - o Settlement
 - o Tilting
 - o Cracking
 - o Separation
- Fences/Walls 6-point percentage scale:
 - o Leaning 'none', <10%, 10-20%, 20-50%, 50-75%, 75-100%
 - o Ruptured
 - o Fallen
- Retaining Walls 6-point percentage scale:
 - o Leaning 'none', <10%, 10-20%, 20-50%, 50-75%, 75-100%
 - o Ruptured

- Fallen

Structural Damage

- External Cladding 6-point percentage scale:
'none', <10%, 10-20%, 20-50%, 50-75%, 75-100%
 - Diagonal cracking
 - Vertical cracking
 - Horizontal cracking
 - Detached/unstable
 - Fallen
 - Corners separating
- Windows 6-point percentage scale:
'none', <10%, 10-20%, 20-50%, 50-75%, 75-100%
 - Jammed
 - Broken
- Doors 6-point percentage scale:
'none', <10%, 10-20%, 20-50%, 50-75%, 75-100%
 - Jammed
 - Broken
- Internal Cladding 6-point percentage scale:
'none', <10%, 10-20%, 20-50%, 50-75%, 75-100%
 - Cracking at corners
 - Joint cracks
 - Separation
 - Nail popping
 - Buckling
- Roof 6-point percentage scale: 'none',
<10%, 10-20%, 20-50%, 50-75%, 75-100%
 - Mis-alignment
 - Buckling
 - Hips loose
 - Popped fixings
 - Joint cracks
 - Tiles dislodged

Foundation Damage

- Overall damage 5-point scale, 'none' to 'severe'
- Concrete perimeter
 - Floor detached from perimeter?
 - Cracking in perimeter (number, size range and any mis-alignment on each side)
- Piles (for concrete perimeter and deep piled foundations) 6-point percentage scale: 'none',

- Settled <10%, 10-20%, 20-50%, 50-
- Tilting 75%, 75-100%
- Slab
 - Joint separation
 - Cracking (width, length and any offset for each visible crack)

Global Settlements	Only recorded if able to measure.
Floor Tilts	Measured with the smart level
Floor Levels	Measured with the Zip Level

4.3.4.2 Ratings Criteria

During the inspections, a variety of data were collected, some of which was measured with the equipment shown in Section 4.3.2 and some of which had to be judged quantitatively by the inspector, as they could not be measured. The ratings criteria used for the 4th September 2010 inspections were taken and expanded on for this phase of the research. Table 4.4 shows the guidelines that were used to determine the ratings for liquefaction and lateral spreading effects and foundation and superstructure damage.

Table 4.4. Severity ratings for each damage index with descriptions for each.

Damage Level	#	Description
Liquefaction Severity		
None	0	No surface manifestation of liquefaction during an event (i.e. sandboils) and no visible heaving/sinking of ground.
Low	1	Small sand boils visible (<2m in diameter & <200mm depth), small amounts of sand ejecta around house.
Moderate	2	Large sand boils visible (>2m in diameter &/or >200mm depth) but not full coverage with ejecta.
High	3	Near full ground coverage with sand ejecta, 50-200mm thick.
Severe	4	Full ground coverage with sand ejecta, >200mm thick.
Lateral Spreading Severity		
None	0	No cracking in ground or evidence of lateral spreading
Low	1	Total width of any cracks on section <20mm
Moderate	2	Total width of cracks >20mm and <100mm
High	3	Total width of cracks >100mm and <300mm
Severe	4	Total width of cracks >300mm
Foundation Damage Level		
None	0	No discernible damage to foundation. Floor levels construction tolerance or less.
Low	1	Low levels of tilting &/or cracking. No separation.
Moderate	2	Moderate levels of tilting or cracking or low levels of both. Some minor separation possible
High	3	Moderate – high levels of cracking and/or tilting. Separation in foundation structure.
Severe	4	Significant tilting, uniform or differential. Large cracking and separation, break-up of foundation structure.
Superstructure Damage Level		
None	0	No damage to structural components. Minor hairline cracking in plaster at corners of windows/doors
Low	1	Cosmetic cracking to interior, hairline cracks in any brickwork. No doors or windows jamming
Moderate	2	Significant cracks interior, cracking in brickwork up to 10mm wide. Doors and windows jamming but not visibly.
High	3	Large cracks interior, 10-20mm cracks in brickwork. Racking resulting in visibly distorted doors and windows
Severe	4	Large structural cracks, separation of structural components. Jammed &/or broken doors &/or windows due to racking.

Figure 4.14 illustrates the criteria used to rate the overall movement of the structure for a number of standard deformation modes. These criteria were taken from EQC inspection forms used following the earthquakes, where they were used as a quick way to assess the seriousness of damage to a house in terms of the estimated repair cost (DBH (2010, 2011) and MBIE (2012)).

Circle any combination of the following

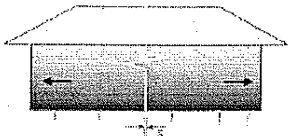

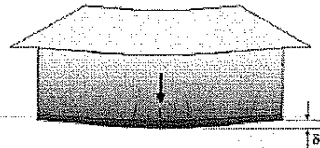
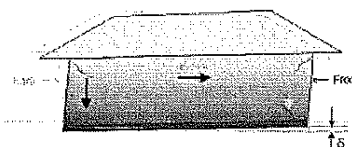
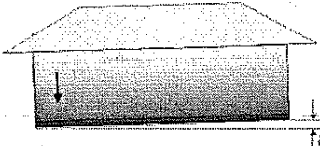
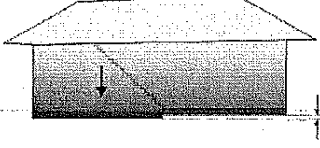
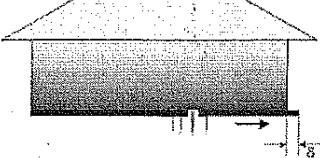
Type \ Severity	Minor <\$10,000	Moderate \$10,000 to \$100,000	Severe >\$100,000
Stretching 	0 to 10 mm	5 to 30 mm	20 to 500 mm ⁺
Hogging 	0 to 10 mm	10 to 50 mm	30 to 500 mm ⁺
Dishing 	0 to 10 mm	10 to 50 mm	30 to 500 mm ⁺
Racking/Twisting 	0 to 20 mm	10 to 50 mm	30 to 500 mm ⁺
Tilting 	0 to 20 mm	10 to 100 mm	30 to 500 mm ⁺
Differential Settlement 	0 to 5 mm	5 to 50 mm	20 to 500 mm ⁺
Differential Displacement 	0 to 20 mm	5 to 100 mm	30 to 500 mm ⁺

Figure 4.14. Ratings criteria of general structural deformation modes, taken from EQC inspection form (DBH (2010, 2011) and MBIE (2012)).

4.4 Ground Conditions

During the detailed house inspections, the manifestation of liquefaction and lateral spreading that occurred in the earthquakes was easily visible at the surface (when liquefaction was of sufficient severity), allowing qualitative judgements to be made on the ground conditions and liquefaction level at each property. However, it is still useful to examine more closely the soil profiles within the suburbs where houses were inspected, in order to gain a better understanding of the characteristic

ground conditions, and any features of the soil profiles which further explain the surface liquefaction and the damage to houses that was observed.

4.4.1 Data Used

To examine the soil profiles in the areas where house inspections were conducted, five CPT logs were chosen from the Canterbury Geotechnical Database (Figure 4.15). The four suburbs where these are located; Ilam, Merivale, Burwood and Dallington, were chosen to cover the range of liquefaction severities seen in the 22nd February 2011 earthquake, and those that were covered in the house inspections, as shown by the UC Liquefaction Map from Cubrinovski & Taylor (2011), overlaid in Figure 4.15. In each area, a number of CPT tests were examined before choosing the few examples analysed here. No attempt is made to provide results that are completely representative of the wider areas where each CPT is located, but rather these logs were selected as they illustrate some typical features of the ground conditions and soil profiles at each location.

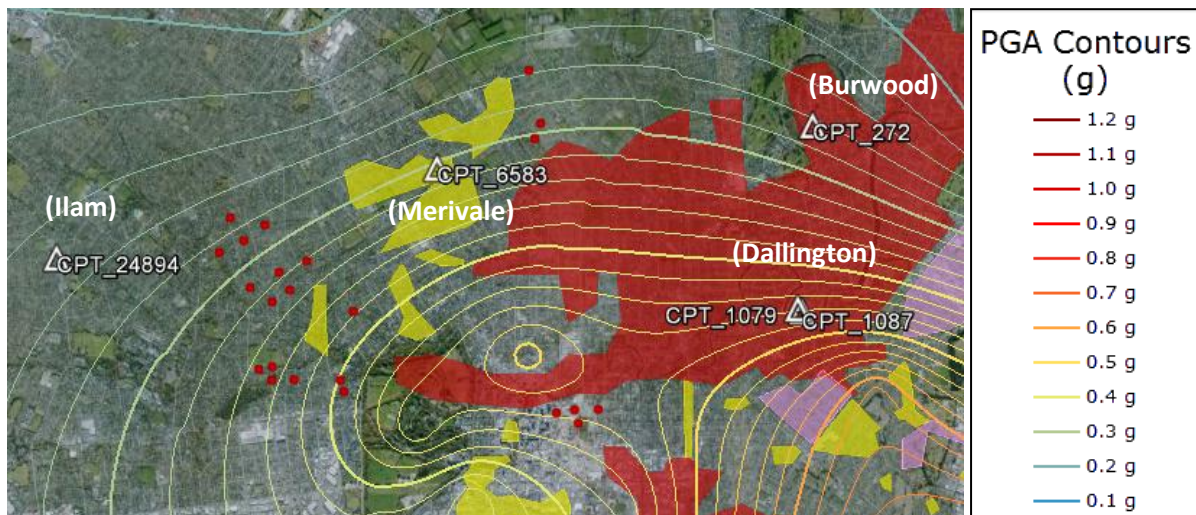


Figure 4.15. Locations of CPT tests used for analysis of liquefaction susceptibility, overlaying the UC Liquefaction Map (Cubrinovski & Taylor, 2011) and peak ground accelerations for the 22nd February 2011 earthquake (O'Rourke et. al., 2012).

Table 4.5. Summary of CPT data with peak ground accelerations and groundwater depths used for liquefaction analysis in each suburb.

Suburb	CPT/s chosen	PGA	GWT
Ilam	#24894	0.25g	3m
Merivale	#6583	0.32g	1m
Burwood	#272	0.26g	0m
Dallington	#1079, #1087	0.45g	0m

Table 4.5 shows the CPT logs chosen for each suburb. It also lists the peak ground accelerations and groundwater table depths used in the liquefaction analysis in CLiq. Approximate values for the peak ground accelerations during the 22nd February 2011 Christchurch earthquake at each CPT location were determined from the PGA contours shown in Figure 4.15, from O'Rourke et. al. (2012).

Groundwater depths were estimated from the Canterbury Geotechnical Database map for median water table depths, which were calculated from the difference between the February 2012 Digital Elevation Model (DEM) (Canterbury Geotechnical Database Map CGD0500) and the median water table elevations from GNS Science (2013). The groundwater level was taken as 0m for Burwood and Dallington. This is due to the proximity to water ways at both of these CPT locations, and the generally high groundwater tables in the East of Christchurch, which were also made higher by the significant global ground settlements that occurred in the Canterbury earthquakes.

4.4.2 Liquefaction Analysis

The raw CPT data was analysed with the liquefaction analysis program CLiq (GeoLogismiki, 2006). This program analyses the CPT data based on the CPT procedure of Youd & Idriss (2001) to provide the cyclic resistance ratio (CRR) and cyclic stress ratio (CSR). It also provides estimates of the soil types present using the procedure of Robertson (1990). The inputs required for this program were:

- The tip resistance, q_c , shaft friction, f_s and the dynamic pore pressure, u_2 throughout the depth of each CPT test.
- The moment magnitude for the 22nd February 2011 earthquake of $M_w = 6.2$.
- Approximate peak ground accelerations (PGAs) at each CPT test site, shown in Table 4.5.
- Groundwater depth (Table 4.5).

The results of the liquefaction analysis for each CPT are shown in Appendix M.

4.4.3 Results

The peak ground accelerations in Figure 4.15 and Table 4.5 show that the 22nd February 2011 Christchurch earthquake produced large seismic demands throughout Christchurch. These, combined with the poor soil conditions and high water table over large areas of Christchurch, which are discussed in the following sections, led to the severe and widespread liquefaction that occurred.

4.4.3.1 Ilam

The CPT chosen in Ilam, number 24894, was located very close to the no-liquefaction inspections which were carried out (shown in Figure 4.13). The soil profile here consisted of a competent crust layer from the ground surface to approximately 3m depth, which is also where the water table was. This was seen fairly consistently through the CPTs examined in Ilam, and was due to the 3m deep water table. Also, the soils in the first 3m were mainly cohesive, non-liquefiable clays and silty clays, with some small lenses of silty sand. This crust layer provides good protection for structures on the surface against any possible liquefaction at greater depths. Below this crust layer was 2.5m of sand and silty sand which was dense enough that it could not liquefy. Together this top 5.5m of non-liquefiable soils shows why no surface liquefaction was seen in Ilam, and why this was chosen as an

appropriate place to conduct the no-liquefaction inspections. The lower PGA here of 0.25g also meant the demands were lower than for some of the central and Eastern suburbs.

The sand and silty sand below the crust layer does not always have a high enough density to be non-liquefiable, with some CPTs showing the possibility of liquefiable sand below the crust, with further lenses of clay and loose, liquefiable sand and silt at greater depths, but due to the competent crust layer there were no liquefaction effects at the surface. For the CPT and liquefaction results see Appendix M.1.

4.4.3.2 Merivale

The ground in Merivale is characterised by layered soil profiles. CPT 6583 which was chosen for Merivale (Appendix M.2) shows a 3.5m crust layer of clay and silty clay, with some thin silt lenses. This is similar to Ilam, and despite the shallower groundwater depth of 1m, still provides a competent crust layer to protect against liquefaction. This crust layer is underlain by 5m of loose, liquefiable sand and silt, with a few thin clay lenses. Below this is 1.5m of clay and silty clay, which is underlain by sand which is generally too dense to liquefy.

The 5m of consistently weak, liquefiable sand from 3.5m to 8.5m depth would suggest that serious effects could be caused by the liquefaction of this layer under the significant demands produced by a PGA of 0.32g. However, the competent 3.5m crust layer protected the surface somewhat from these effects, which is why only moderate levels of surface liquefaction ejecta were seen in Merivale (Figure 4.15).

4.4.3.3 Horseshoe Lake, Burwood

The CPTs examined in the Horseshoe lake area in Burwood showed very poor soils which were highly susceptible to serious liquefaction. There is less than 1m of fine-grained crust soil in CPT 272 (Appendix M.3), which is underlain by a full 7m of loose, liquefiable sand and silty sand, with only one thin clay lense. Below 8m the sand and silty sand is much denser, being basically non-liquefiable, with a couple of thin clay lenses, down to a depth of 24m. The 7m of loose sands is highly liquefiable, even under the lower PGA of 0.26g in Burwood which produced a smaller CSR demand relative to Merivale and Dallington. The poor soil conditions are compounded by the groundwater table being at the ground surface. When liquefied, a continuous layer of this thickness could easily break through the thin crust, directly affecting structures such as residential houses founded within the top 1m of soil, and has resulted in the extensive liquefaction seen in Burwood, particularly in Horseshoe lake.

4.4.3.4 Dallington

The soil conditions found in Dallington are similar to those in Burwood, and are shown by the two CPTs 1079 and 1087 (Appendix M.4). CPT 1087 shows a similar crust layer to 0.8m depth as was seen in Burwood, while CPT 1079 has no crust. In both CPTs this is then followed by very loose sand

and silty sand to between 6-6.5m, with a tip resistance of less than 8MPa in both CPTs. Below this, the sands become denser and some thin clay lenses are seen. The weak sand layers shown in both CPTs are very susceptible to liquefaction, which again is compounded by having the groundwater table at the ground surface. Dallington also has a PGA of 0.45g, higher than Burwood. These ground conditions resulted in the severe liquefaction ejecta seen in Dallington, and the high levels of damage to residential houses. This is due to the consistent depth of liquefaction, the water table at the ground surface and the absence of any competent crust layer to isolate structures founded on the surface.

4.5 Deformation Types

The types of damage observed in these detailed inspections following the 22nd February 2011 earthquake were the same as the typical damages observed for houses following the 4th September 2010 earthquake, so they won't be repeated here. These are shown in Chapter 3, Section 3.4 which illustrates the typical damages for the different aspects of each property that were recorded, such as the superstructure, both interior and exterior, damage to different foundation types, some structural damage, and typical damage to the surrounding property. Houses with ribraft and deep piled foundations showed the same types of damage as houses with slab-on-grade foundations. This is because to outside appearances, these three foundation types are all concrete slabs, and so visible damage to these are all the same. The deformation modes of each are also similar, and will be discussed in this section.

There were a number of different overall deformation modes in the houses inspected. These related to the severity of structural damage suffered and to the deformation modes shown in Figure 4.14, but are more specific to the deformation of the foundations based on their floor levels. Some were common to all foundation types, while some were only seen in houses with one foundation type. Some modes represent only the minor end of the structural damage scale, while others were seen from low to severe levels of structural damage. Each house that was inspected was given one or sometimes two of these deformation types, along with a rating of the level of damage.

4.5.1 *Low to Moderate Deformation Modes*

These modes of deformation were observed in all foundation types inspected for this research, and were found in houses at the lower end of the damage scale.

4.5.1.1 Construction Tolerances/Instrumental Error

This deformation mode was assigned to houses with any foundation type where no real evidence of damage was found, but where the instruments indicate some small tilts which can be attributed to other factors than structural damage. These could be due to construction tolerances, as newly constructed foundations never have completely flat floors, as discussed in Chapter 2, Section 2.5.1.7.

They can also be due to changing floor coverings between measurements or instrumental inaccuracies, as noted in Table 4.3. The combination of these three sources of measurement inaccuracies give instrument readings of $0.1\text{--}0.2^\circ$ on the smart-level, and $\pm 5\text{mm}$ on the Zip Level (see Figure 4.16). Houses given this deformation mode are always assigned a damage rating of ‘none’, and generally show no or low damage to their superstructures and foundations.

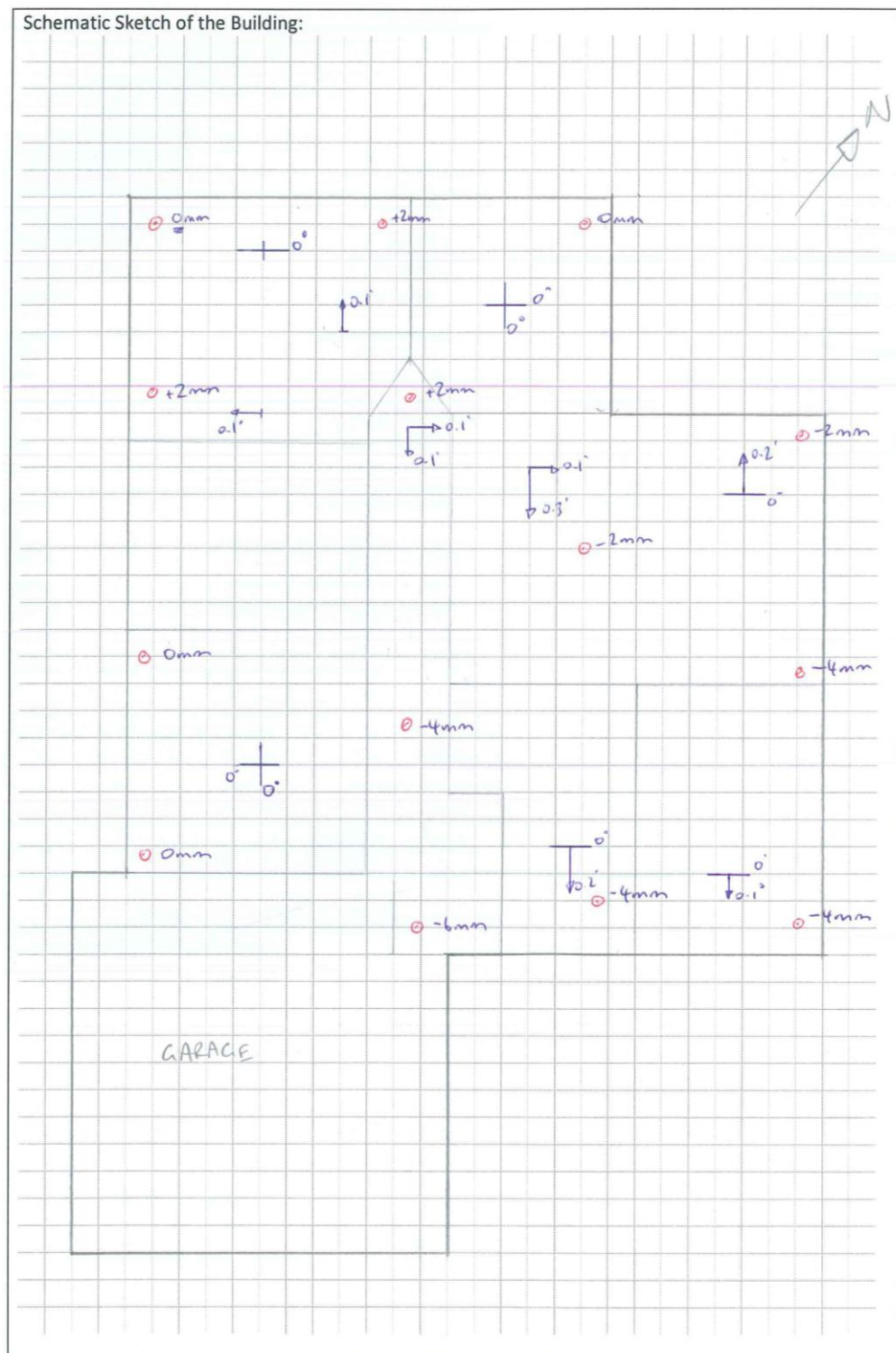


Figure 4.16. Example floor plan for a house with construction tolerance movements.

4.5.1.2 No Distinct Damage Mode

This deformation mode is the next level up from construction tolerances. It is assigned to buildings where the instruments indicate some minor localised tilting, and there is no noticeable uniform tilt direction. These can be 0.1 - 0.4° on the smart-level, and $\pm 20\text{mm}$ on the Zip Level. Houses given this deformation mode are generally given a 'low' damage rating, and show no or low, sometimes moderate damage to their superstructures and foundations. An example floor plan is shown in Figure 4.17.

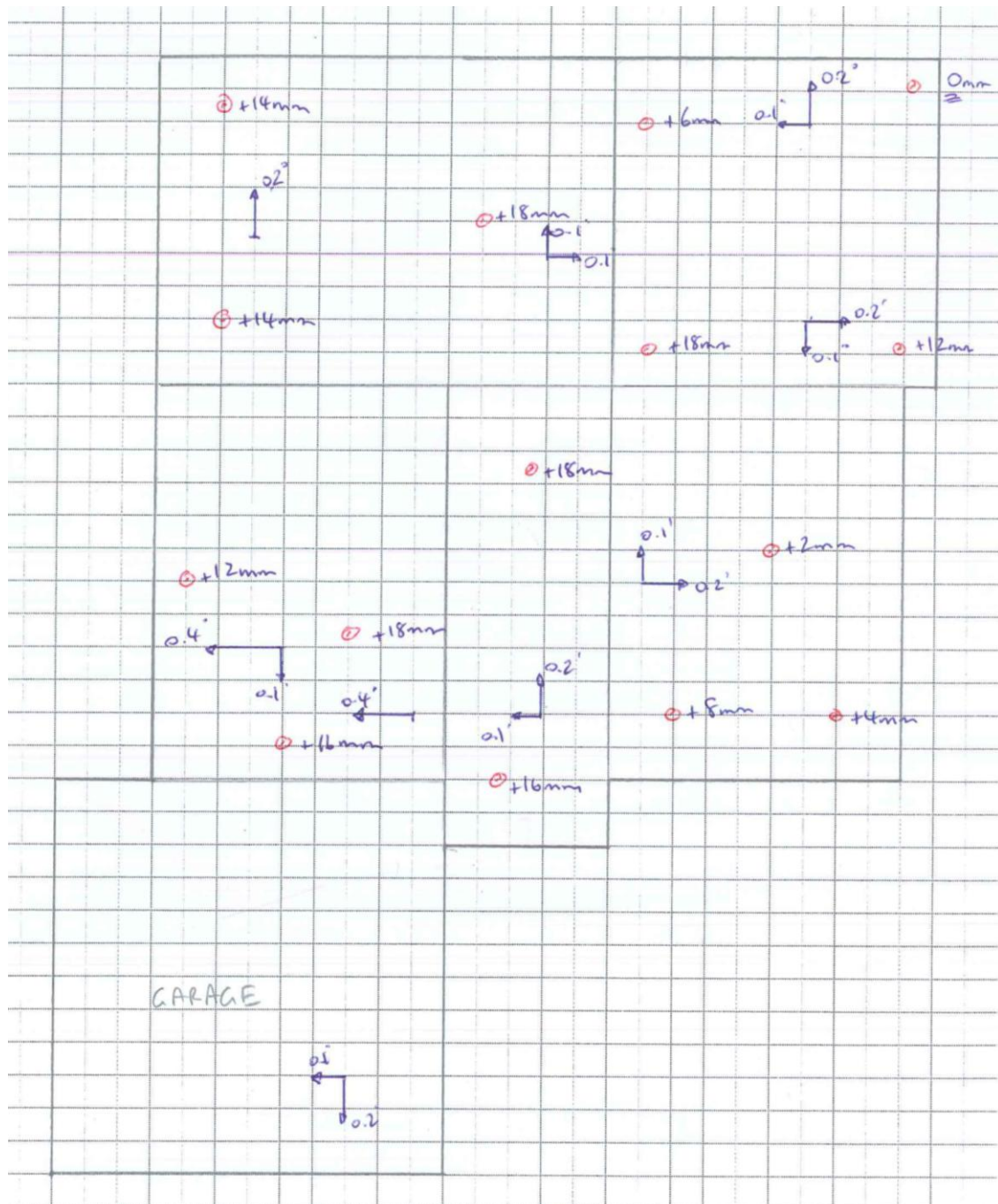


Figure 4.17. Example floor plan with measurements indicating 'no distinct damage mode'.

4.5.1.3 Complex Tilting

This deformation mode is assigned to foundations with tilting too large to be considered minor, but can't be classed as another more distinctive type of tilting such as uniform, hogging or dishing. Tilt measurements range from 0.1 - 0.7° on the smart-level, and $\pm 50\text{mm}$ on the Zip Level. Buildings with this deformation mode are given low, moderate or high damage ratings and can exhibit low to high levels of superstructure and foundation damage. An example floor plan is shown in Figure 4.18.

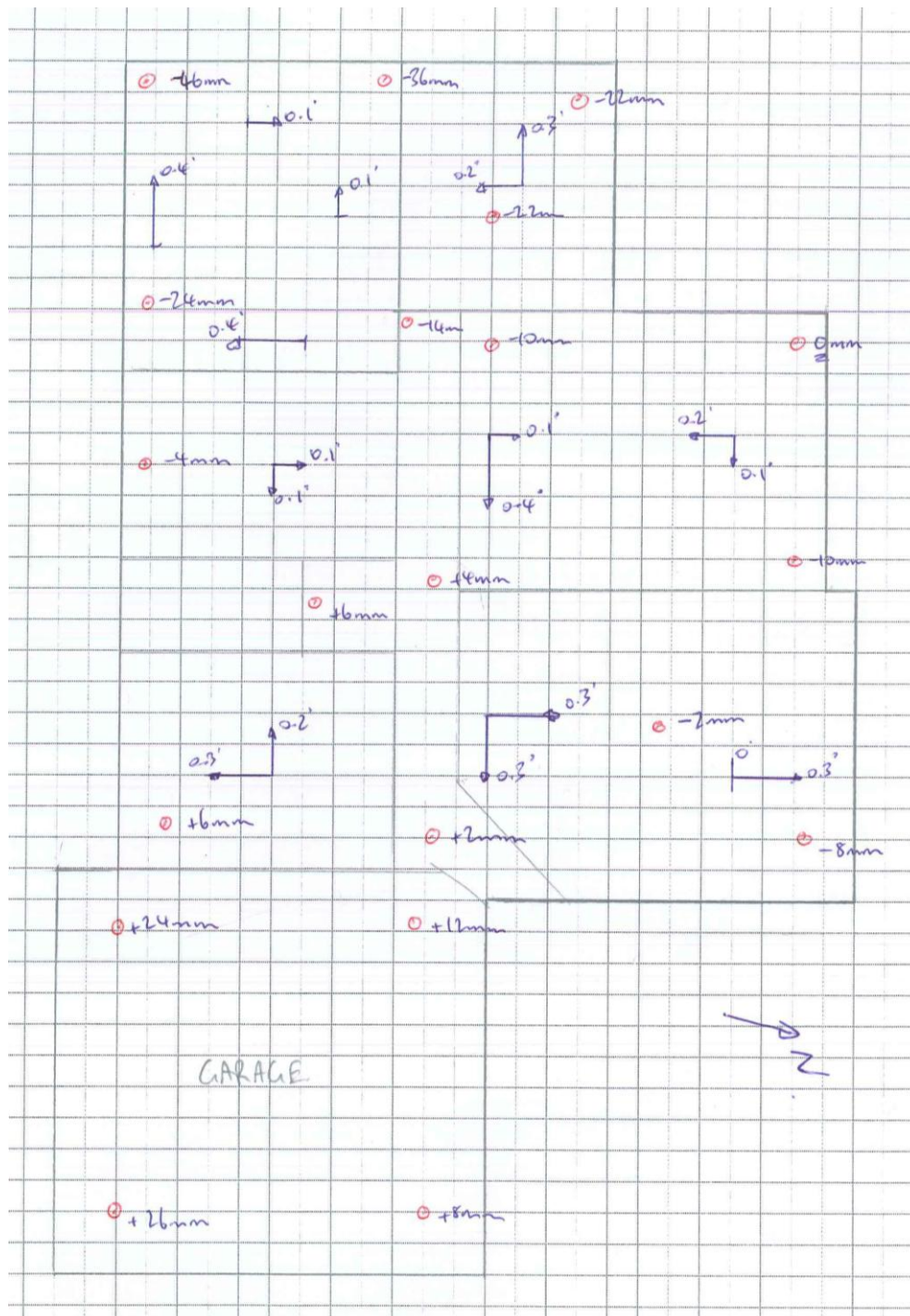


Figure 4.18. Example floor plan showing measurements for complex tilting.

4.5.2 Severe Deformation Modes for all Foundation Types

These deformation modes were also seen in houses with all four foundation types. Houses experiencing these deformation modes can suffer the full range of superstructure and foundation damage. They are also assigned a rating for the deformation mode which can be from low to severe, with differential settlements ranging from 10-20mm and 0.1° , up to $>200\text{mm}$ and $>2^\circ$ of floor tilt.

4.5.2.1 Uniform Tilting

Uniform tilting is where the house is tilting generally in one direction. This can be towards a corner diagonally, or along one wall or major axis of the house. Major axis uniform tilting was the most common, as in this direction the structure as a whole is stiffer than diagonally. Diagonal differential settlement is more often associated with folding or broken tilting, discussed below. To be strictly uniform tilting, there must be no hogging or dishing of the foundation within the footprint. However, cases were seen, particularly for concrete perimeter foundations, where the overall structure was tilting uniformly (i.e. along the perimeter), but there was still local humping or dishing in the floor. This was assigned as overall tilting of the structure, but was combined with complex tilting or some other mode, depending on the local deformations in the floor. Structural damage to the foundation and superstructure can be lower than for other deformation modes at the same damage level. This is because the uniform tilting places less stress on the structure in terms of racking and twisting, much like uniform settlement. Figure 4.19 shows an illustration of uniform tilting along a major axis.

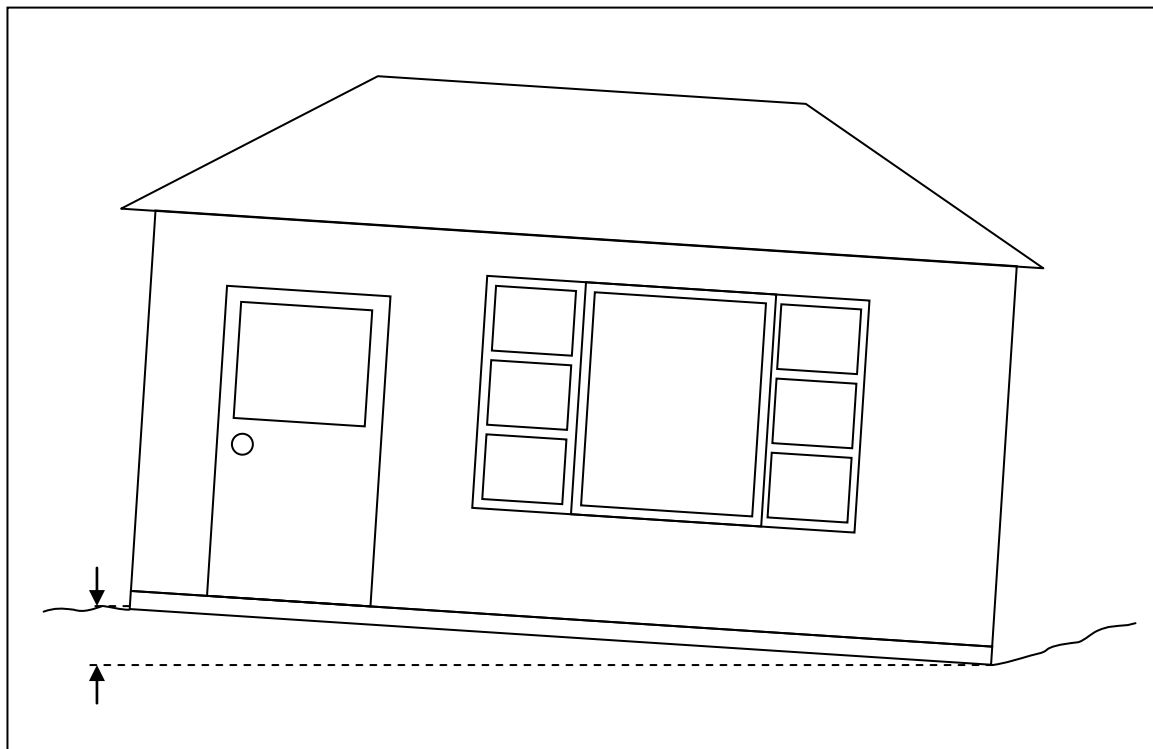


Figure 4.19. Uniform tilting of a house along a major axis.

4.5.2.2 Hogging

Hogging is typically where the centre of the building has raised up above the perimeter, or the perimeter has sunk down more than the interior, or a combination of both. It can be caused by the upwelling of sand ejecta beneath the inner part of the foundation, pushing it upwards, or by the extra structural weight on the perimeter of the foundation causing it to settle further under loss of bearing pressure. To be classed as hogging, the interior floor must be raised over a significant part of the house (i.e. generally more than one room). If it is less than this, the deformation may be classed as no distinct mode, or complex tilting, depending on the severity. Hogging can take a multi-directional form where just the centre of the foundation is raised, with all sides lower, or it may be planar, raised across the middle, as shown in Figure 4.20, with only two sides down, where cross-sections through the house all show the same profile. Damage to the superstructure is caused by significant racking and differential movement damage in this deformation mode, as shown in Figure 4.20 for the doorway.

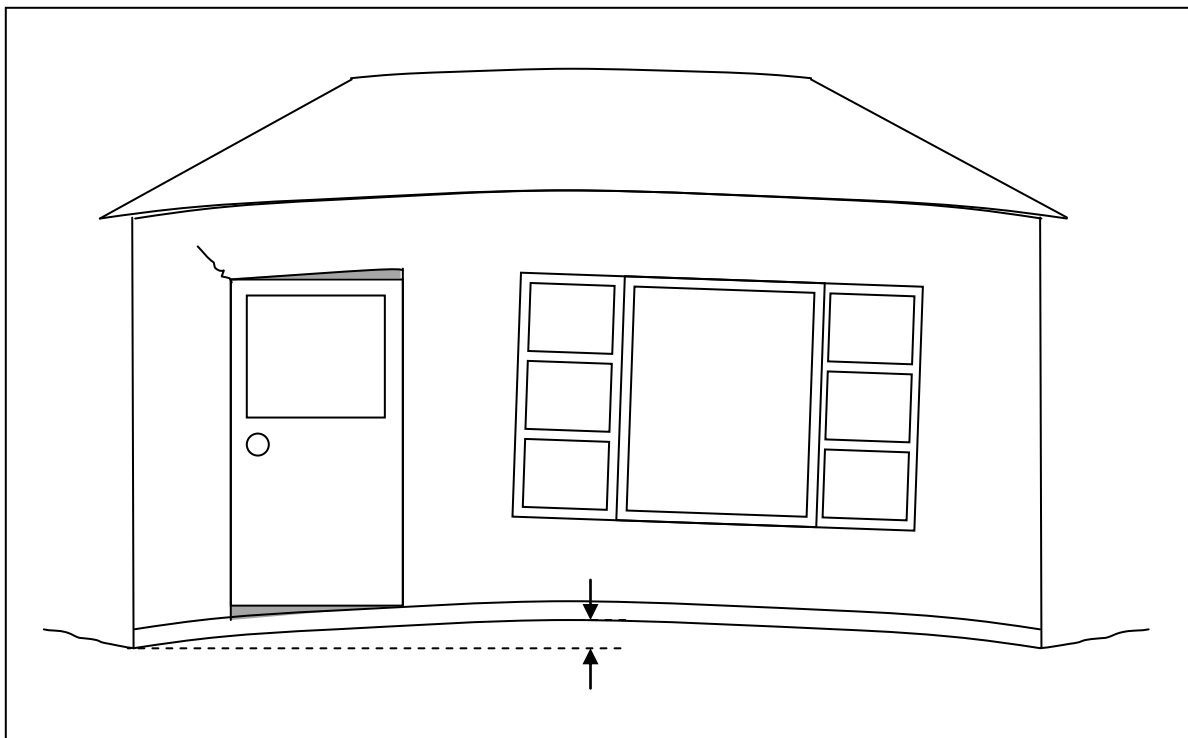


Figure 4.20. Hogging of building, showing racking damage to door frame.

4.5.2.3 Dishing

Dishing is the inverse mode of hogging. It is where the centre of the building settles more than the perimeter. This can be caused by having a greater weight in the centre of the building (i.e. a second storey, or a large solid brick or concrete chimney), or a more severe loss of bearing capacity there. It can also occur in deep piled foundations where only the perimeter is piled, giving it more resistance to settlement than the centre of the building. To be classed as dishing, the interior floor must be settled relative to the perimeter over a significant part of the house (i.e. generally more than one room). If it is less than this, the deformation may be classed as no distinct mode, or complex tilting, depending on

the severity. As for hogging, dishing can be multi-directional with just the centre settled, or planar, as shown in Figure 4.21. Superstructure and foundation damage is again caused by the differential movement of the foundation.

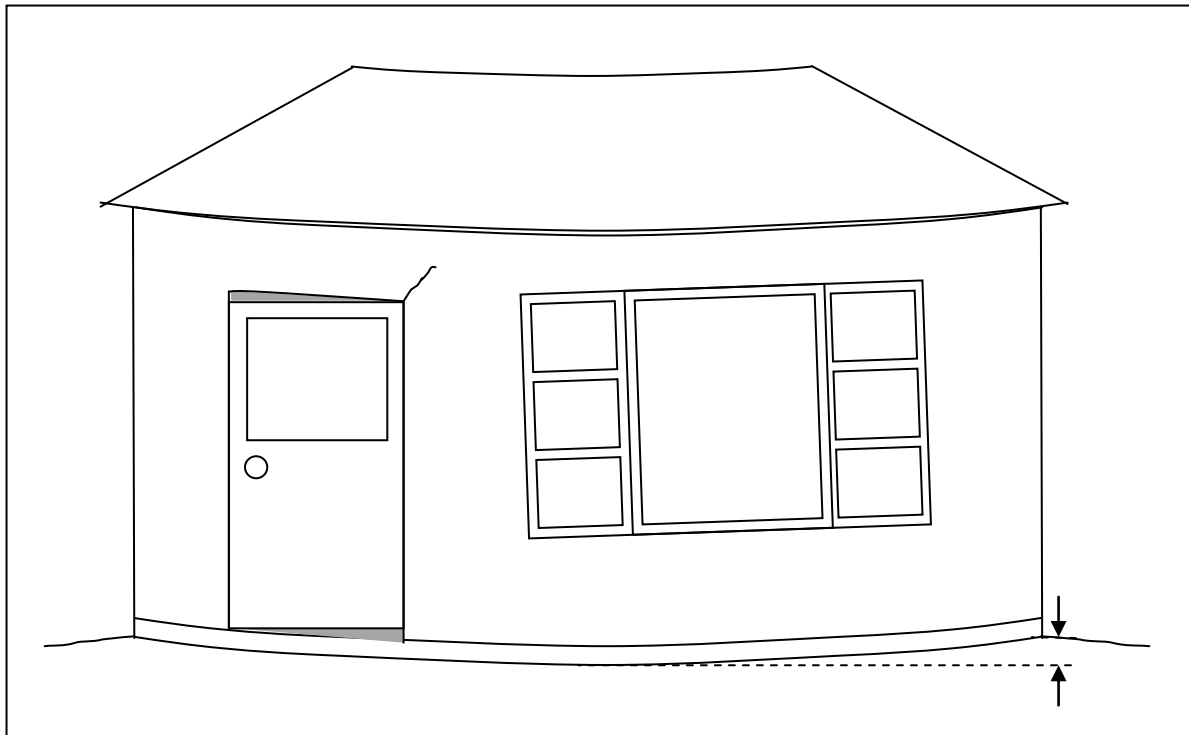


Figure 4.21. Dishing of building, showing racking damage to door frame.

4.5.2.4 Folding

This deformation type can occur in all foundation types, but is more commonly seen in slab-type foundations. Folding is where 2-directional tilt occurs along a line through the foundation, similar to holding a piece of paper at two points and letting it fold downwards (or upwards). Figure 4.22 shows folding along a diagonal line through the foundation. This deformation is similar to dishing and hogging, however it occurs along a line, rather than in an area, and is more specific to slab-type foundations. As for hogging and dishing, superstructure damage is caused by racking and the differential deformation of the foundation.

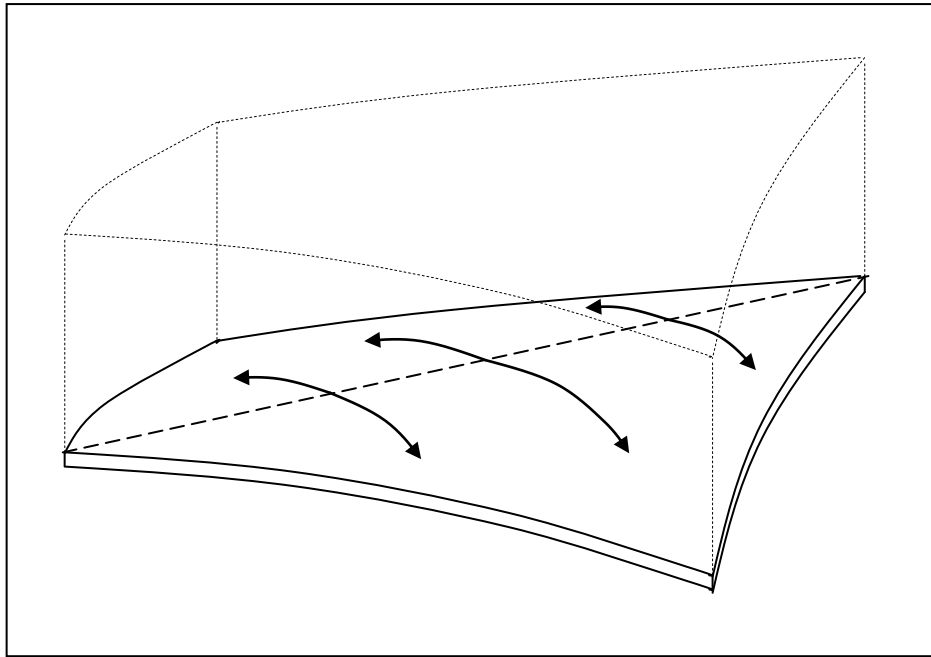


Figure 4.22. Folding shown for a slab foundation.

4.5.3 Severe Deformation Modes for Specific Foundation Types

These deformation modes were only seen in specific foundation types, due to the unique structural and deformation characteristics of each one. As for the other severe damage modes, the full range of superstructure and foundation damage is again seen. The deformation mode is given a rating from low to severe, and as for the other severe deformation modes, differential settlements can range from 10-20mm and 0.1° , up to >200mm and $>2^\circ$.

4.5.3.1 Broken Tilting

(Slab-on-grade, RibRaft, Piled)

Broken tilting occurs only in slab-based foundations, and more commonly in plain slab-on-grade foundations, due to the weaker slabs. It is where irregular sections of the building plan (i.e. a garage or room that protrude from the main body of the floor plan) break away from the main body, and tilt into, or away from the centre. The settlement can be due to different levels of bearing loss under the footprint of the house, or lateral spreading under an irregular part of the house, causing it to settle and tilt away from the body of the house, or it can be due to loss of bearing under the centre of the house, where this settles more, and the irregular parts tilt in towards the centre. This can be exacerbated if part of the house is 2-storey, as the added weight increases the settlement of the 2-storey section. As the flexural capacity of a basic slab-on-grade foundation built to NZS3604 is quite low, the slab breaks, allowing different levels of settlement in the different parts of the house. Structural damage is often high with this mode (provided the level of deformation is also high), as the differential settlement and break-up of the foundation can cause large cracking and separation in walls and ceilings. Figure 4.23 shows broken tilting of the garage of a standard slab-on-grade foundation house,

with Figure 4.24 showing how the two protruding wings of the house, the garage and master bedroom, both tilted inwards due to larger settlement of the main body of the house.



Figure 4.23. Broken tilting of the garage of a slab-on-grade foundation house, elevation view.



Figure 4.24. Broken tilting of garage and master bedroom due to higher settlement of body of house, shown in plan-view.

4.5.3.2 Room Humping

(Concrete Perimeter)

Localised room humping is a mode of deformation unique to concrete perimeter foundations with short interior piers. Differential settlement between the concrete perimeter and the interior piles causes humping of the floor in the middle of rooms in the house, shown in Figure 4.25. It can be caused by two situations. The exterior perimeter supports the exterior walls and the roof load transferred to the external walls. These are often the heaviest part of the house superstructure, and cause larger loads on the concrete perimeter than the interior piles, which only support floor and live loads, and some interior walls. This causes the perimeter to settle further than the interior piles, and the majority of the superstructure settles with it, as it is effectively only supported by the perimeter. This leaves the piles supporting the floors with less settlement, producing the floor humping. This often also results in racking of the superstructure locally, causing doors and windows to jam. The other cause can be when large amounts of sand ejecta push the piles and floor up from underneath the house. This mostly occurs in conjunction with the settlement of the perimeter. Superstructure damage caused by this deformation mode will generally be less than for hogging, dishing or overall humping, as the deformation is confined to one room, and mostly just affects the floor, limiting the damage done to the structure above it.

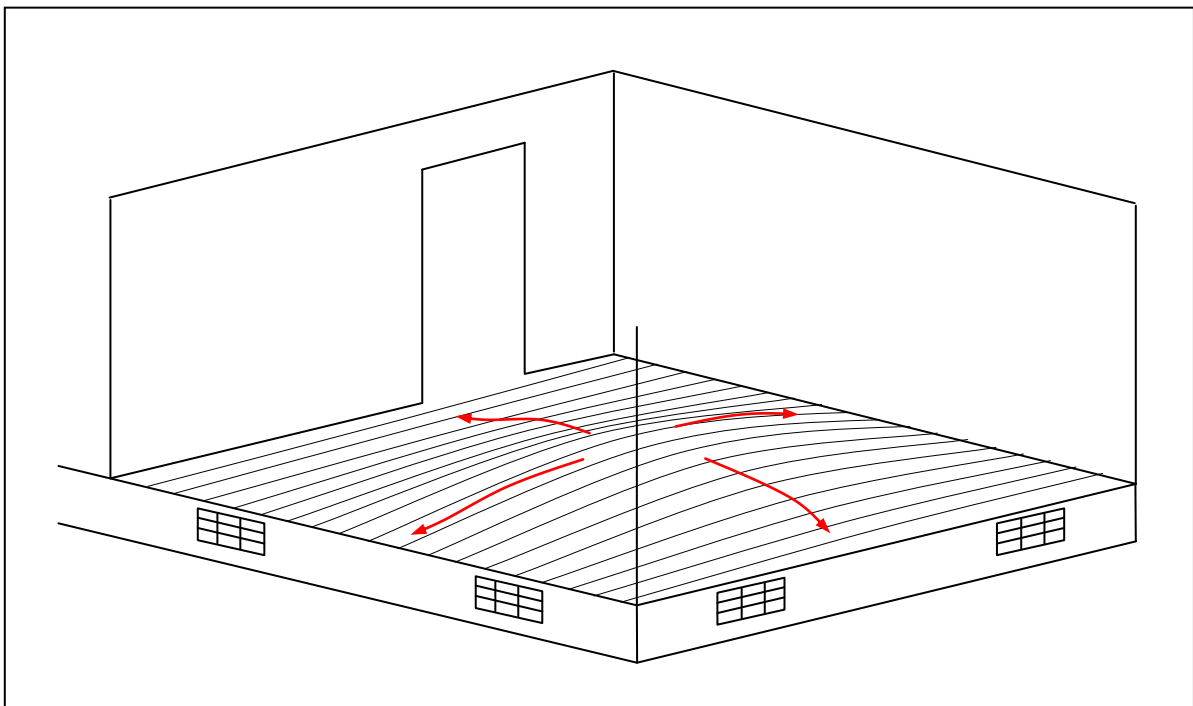


Figure 4.25. Room humping in a perimeter foundation house.

4.5.3.3 Overall Humping

(Concrete Perimeter)

Overall humping occurs only in concrete perimeter foundations with short interior piles. It is similar to localised room humping, the difference being that the humping and settlement of the perimeter is not localised room-by-room, but occurs over the whole house footprint. Some of the piles may settle with the perimeter, while some of the perimeter may not sink as much. The same effects as for room humping are generally still seen, with humping of floors creating large tilt angles and racking of the superstructure causing jamming of doors and windows. Interior brick fireplaces, which are founded on concrete pads, are another location with high foundation loads, meaning these often settle more than the surrounding floors, causing humping around them. Superstructure is generally higher than for the equivalent level of room humping, as the superstructure is directly affected by interior walls also being pushed up with the floor, causing racking and differential displacement, as shown in Figure 4.26.

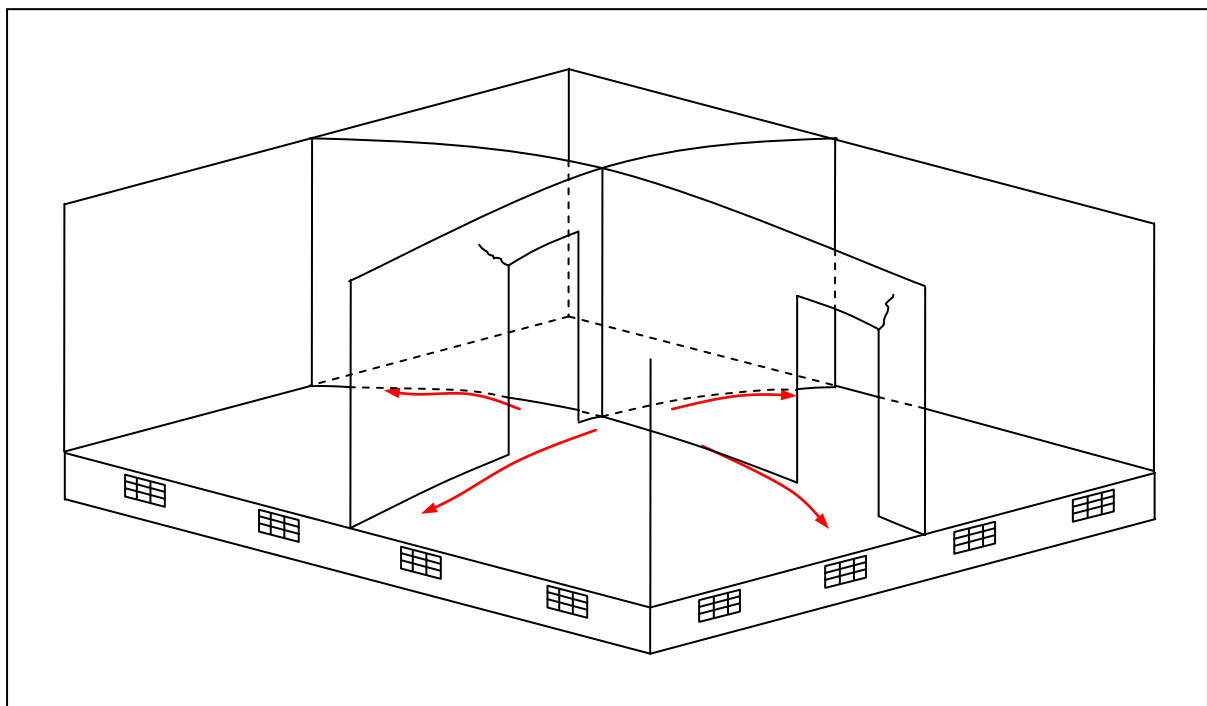


Figure 4.26. Overall humping of a concrete perimeter foundation house.

4.6 Data Analysis

4.6.1 Data Organisation

The data collected during the inspections was organised into a large spreadsheet, with separate spreadsheets used for each foundation type. All the values for the parameters listed in Section 4.3.4.1 were included in the spreadsheet and the measured values, including the slopes and floor levels, were also added.

4.6.1.1 Data Conversion

The first step in the data analysis process was to convert the data in the spreadsheets to number formats for graphing. Damage indices that were given ratings of ‘none’, ‘low’, ‘moderate’, ‘high’ and ‘severe’ were assigned number values of 0 to 4 respectively, as shown in Table 4.4. This was done for indices such as liquefaction, lateral spreading, foundation and superstructure damage and the overall damage modes taken from the EQC assessment forms. Number values were also assigned to all the damage indices that were given percentage values of ‘none’, <10%, 10-20%, 20-50%, 50-75% and 75-100%. They were given ratings of 0 to 5 respectively.

4.6.1.2 Data Quality Assurance and Censoring

Once the data were organised, they were scrutinised to identify any outliers, unreliable data and any unusual factors that appeared to be affecting the results. Any houses that did not fit in with the main body of the data were removed. This was not such a big issue with the data for this study, as the houses were specifically chosen to first meet the criteria for the different foundation types, and also, where possible, the need for a reasonable spread of land damage. All the buildings inspected were also chosen to be standard one or two-storey houses or town houses where necessary.

For piled foundations, it was decided that as there were only six houses inspected that didn’t have foundations consisting of a concrete slab with piles underneath, these would be removed. This was because their behaviour would have been different to the piled-with-slab combination.

A small number of outliers were identified in the data that had suffered large levels of structural damage for relatively low levels of liquefaction severity. Where this damage was found to be due to significant lateral spreading, and for the inspected houses where extreme levels of lateral spreading were observed (i.e. moderate to severe lateral spreading in multiple earthquakes), the house was removed from liquefaction analysis. Houses that suffered low to moderate lateral spreading which did not seem to adversely affect the results were kept in the data set. Only five slab-on-grade, one ribraft and two piled foundation houses were removed from the dataset because of this.

There were only a small number of houses inspected that suffered significant levels of lateral spreading. This meant that trends in the performance of foundations under lateral spreading could not

be established. However this small number of houses was examined alongside the liquefaction data to provide a comparison between the two, and allow the range of damage under significant lateral spreading to be seen.

4.6.2 Indices Used for Analysis

Once the data from the inspections had been organised into spreadsheets, and converted to numerical formats, a number of new damage indices were added, both from the recorded data itself, and outside sources. This section outlines the new indices that were added, and the indices chosen from the extensive dataset to use in analysis.

In general, it was decided it was better to use more simplified damage indices, those with a smaller range of values, as this allowed simpler, but more robust trends to be identified and conclusions to be drawn. The spread in results for much of the more complicated data was far too great to draw any significant conclusions.

4.6.2.1 Land Damage Indices

In addition to the land damage ratings recorded in the inspections, land damage indices from the EQC ProjectOrbit database were added, as for the 4th September 2010 earthquake inspection data. The following land damage indices were used in the analysis of the data:

- **Maximum Liquefaction Damage.** This was the highest level of liquefaction suffered by a house in any seismic event, using the inspection ratings from Table 4.4. The maximum liquefaction event was chosen as this best represented the severity of ground damage felt by each house. Combinations of the land damage data were explored, but they created unrealistic spread in the data.
- **Liquefaction and Lateral Spreading Observations.** These were taken from the ProjectOrbit database, and were collected through the on-foot rapid inspection of individual properties in residential areas following each earthquake. Observations were categorized according to the quantity of ejected material observed on the ground surface and according to the presence or absence of evidence of lateral spreading. This index was used to verify the liquefaction values recorded in these inspections (i.e. the Maximum Liquefaction Damage values). For more information see Chapter 3, Section 3.5.3.1.
- **Vertical Ground Surface Movements.** These are also from the ProjectOrbit database and give the vertical elevation changes between LiDAR sets taken before and after each major earthquake that approximate the vertical ground movement during that event. This index was used as an additional ground damage indicator, because, as described in Chapter 3, Section 3.5.3.2, free-field ground settlement is directly related to liquefaction severity.

4.6.2.2 Foundation Damage Indices

In each inspected house there were multiple values measured for the floor slope and floor level. These multiple values allowed the assessment of the overall deformation of the house and foundation, but are hard to compare to each other with significant numbers of inspections due to their spatial nature. It is more useful to have a single value per house for graphing and analysis purposes. For this reason, a number of single value parameters were created from the measured data, to go along with the overall foundation damage rating recorded in the inspection. The final indices used for foundation damage were:

- **Foundation Damage Rating.** This was the overall foundation damage as judged by the inspector, based on the criteria in Table 4.4. This was used to give a general idea of foundation damage, and to allow correlation back to the data from the 4th September 2010 earthquake in Chapter 3.
- **Maximum Overall Differential Settlement.** This was the maximum elevation difference over the whole foundation from the zip level floor measurements. It was used as it indicates the overall deformation of the foundation, and provides a comparison to the differential settlement limits set by MBIE (2012), shown in Chapter 2, Figure 2.20. The maximum differential settlement is only a measure of the maximum elevation difference, the way in which the foundation accommodates this difference can be through uniform or non-uniform tilting, illustrated in Figure 4.27. The non-uniform tilting case is much more damaging to the foundation and superstructure, as it can result in racking, hogging and dishing, as described in Section 4.5.

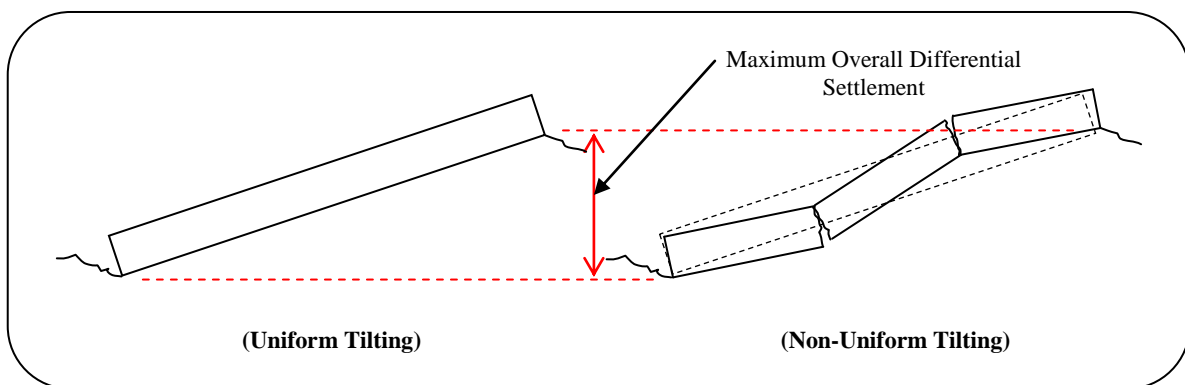


Figure 4.27. Schematic showing the difference between uniform and non-uniform foundation tilting, for identical Maximum Overall Differential Settlements.

- **Maximum Overall Floor Slope (Equivalent Slope).** This was calculated from the maximum overall differential settlement and the horizontal distance between the two measurement points that gave this measurement, read from Google Earth. It was also used as a measure for the overall deformation of the house, like the maximum overall differential settlement, but is normalised by the size of the building.

- **Maximum Floor Slope (Maximum Slope).** This was calculated as for the maximum overall floor slope, with two points from the zip level measurements and the distance between them, however the two points were chosen so as to give the maximum possible slope of the floor that was measured between any two points. A more localised floor slope measurement, this was used to quantify the maximum distortion of the floor.
- **Maximum Floor Tilt (Maximum Local Slope).** This was the maximum tilt angle of the floor recorded over the 1.2m length of the smart level in any direction. The most localised of the slope indices, it indicates local distortion of the foundation.

The four measured foundation damage parameters are shown in Figure 4.28. By definition, as shown in this image, the more localised measurements of floor slope have a minimum possible value of the next most localised measurement, and will often read higher than this, due to the deformation of floors being highly non-uniform.

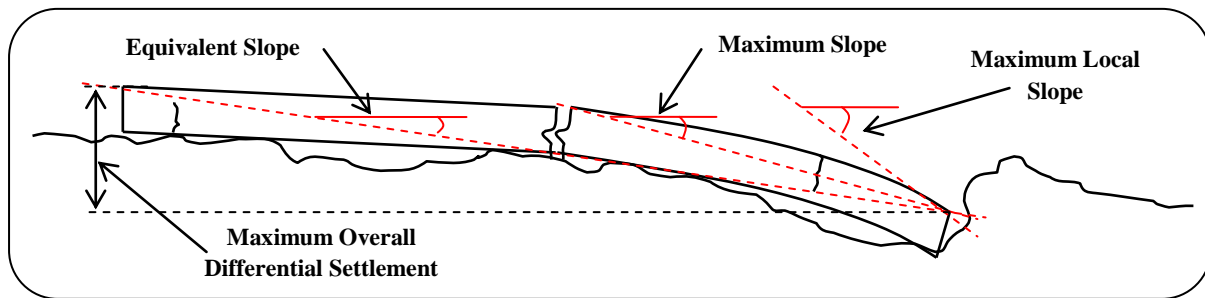


Figure 4.28. Measured foundation damage parameters, shown on a broken slab foundation.

4.6.2.3 Superstructure Damage Indices

The only index used in analysis for superstructure damage was the rating recorded by the inspector during inspections, based on the criteria in Table 4.4. Unlike for the foundations, no measured quantities were recorded that translated to a rating of the level of superstructure damage, as the superstructure was not the main focus of this research. More complicated single figure values were experimented with, including combinations of the different superstructure damage ratings for claddings and deformation type. However these indices were too complicated, creating too large a spread in the data, and didn't have any physical meaning, so were not used. As already discussed, the more simple index allows more robust trends to be explored.

4.6.3 Analysis

With the decisions made of which parameters to use, graphs were produced to allow the analysis of the performance of the different foundation types and the influence of other structural features on this performance. To conduct the analysis, a wide range of graphs were created, comparing the different damage parameters and organised by different structural features, in different types of plots. Table 4.6 shows a summary of the different graphs that were produced.

Table 4.6. Summary table of graphs produced to compare different parameters.

	y-axis									
	Ground Damage			Foundation Damage					Structural Damage	
x-axis	Maximum Liquefaction Damage	Liquefaction/Lat. Spread. Observations	Vertical Ground Movements	Max. Overall Differential Settlement	Equivalent Slope	Maximum Slope	Maximum Local Slope	Overall Foundation Damage	Overall Superstructure Damage	Structural Damage Rating
Maximum Liquefaction Damage				X	X	X	X	X		
Liquefaction/Lat. Spread. Observations				X	X	X	X	X		
Vertical Ground Movements				X	X	X	X	X		
Max. Overall Differential Settlement				X	X	X	X	X		
Equivalent Slope										
Maximum Slope										
Maximum Local Slope										
Overall Foundation Damage									X	X
Overall Superstructure Damage										
Structural Damage Rating										

A number of different plots were produced for each graphing combination shown in Table 4.6, for different combinations of the following structural features:

- Foundation type
- Age of construction
- Heavy and Light claddings
- One and two storeys
- 'Heavy' or 'Light' houses (Heavy = heavy cladding, two storeys or both, Light = one-storey, light cladding)
- Plan shape

Combinations of these structural features were also plotted, in order to isolate the feature being examined from the influence of others, for example, old and new houses plotted for heavy and light combinations.

The types of graphs produced included:

- Bar Graphs. As for Chapter 3, bar graphs were used to show trends in data where no continuous measured data was available. For example, to show the distribution of superstructure damage at each level of foundation damage, where both of these indices had discrete values recorded in the inspections.
- Scatter Plots. Scatter plots were used for combinations between continuous and discrete indices, such as the three floor slope indices plotted against liquefaction damage or the ground settlement from LiDAR data. These allow closer examination of the individual data points, and also included the 'no damage' comparison values outlined in Section 4.6.4 below as baseline reference values for floor slopes and differential settlements.
- Box and Whisker. Box and Whisker plots were used to show the characteristics of the floor slope data at different liquefaction levels. The up and down bars show maximum and minimum values of floor slope at each liquefaction severity, with the 25th and 75th percentiles opening and closing the box, and median values also shown. Where there were insufficient data points for the 25th and 75th percentiles to have any real meaning, only up and down bars and the median were shown. The median values were also used in the scatter plots of floor slopes against liquefaction severity as a reference.

4.6.4 'No Damage' Comparisons

A number of baseline comparisons were used when plotting the different foundation damage parameters listed above, to provide a reference level for the foundation damage being shown. They included values taken from a variety of sources, including the guidance from MBIE (2012) on repair and rebuild of houses, previous research in the field, and the 'no-liquefaction' inspections conducted for this research. The following floor slope and differential settlement benchmarks were used to compare to the inspected data:

- Japanese criteria for the level of damage to houses, following the 2011 Great East Japan Earthquake (Yasuda et. al. (2012))
 - 'Partially destroyed' building: 1/100
 - 'Large-scale, partially destroyed': 1/60
 - 'Completely destroyed': 1/20
- Department of Building and Housing/Ministry of Business, Innovation and Employment surveys of new buildings and damage limits (DBH, 2010, 2011 and MBIE, 2012)
 - Minimum observed local slope (over 2m) in new concrete floors: 1/300
 - Maximum observed local slope (over 2m) in new concrete floors: 1/150
 - Maximum observed overall slope in new concrete floors: 1/500
 - Threshold for 'no damage' to a foundation: 1/200 or deviation in floor level of <50mm
 - Indication of repairs required to a foundation: deviation in floor level of 50-100mm
 - Indication of foundation rebuild being required: deviation in floor level of >100mm
- Inspections of 19 houses in Christchurch, shown in Figure 4.13, which suffered no liquefaction effects in the earthquakes
 - Concrete Perimeter foundations
 - Average Equivalent Slope of all inspections: 1/450
 - Average Maximum Slope of all inspections: 1/210
 - Average Maximum Local Slope of all inspections: 1/100
 - Slab-on-Grade foundations:
 - Average Equivalent Slope of all inspections: 1/720
 - Average Maximum Slope of all inspections: 1/350
 - Average Maximum Local Slope of all inspections: 1/160

4.7 Results

4.7.1 No-Liquefaction Inspections

4.7.1.1 Foundation Damage

Figure 4.29 and Figure 4.30 show the three different floor slope indices for the inspections of houses that did not suffer any liquefaction effects in Christchurch (shown in Figure 4.13), for concrete perimeter and slab-on-grade respectively. For both foundation types, as the floor slope measurement becomes more localised, the values become larger and more varied. The equivalent slope and maximum slope values show a narrow range when compared to the maximum local slope range. This was also seen in the main body of inspections, and will be discussed later, but it is noted here that the trend appears irrespective of the presence of liquefaction.

For each different floor slope index, the concrete perimeter foundation has higher average values than the equivalent for slab-on-grade foundations. This was again echoed in the general inspections and will be discussed later.

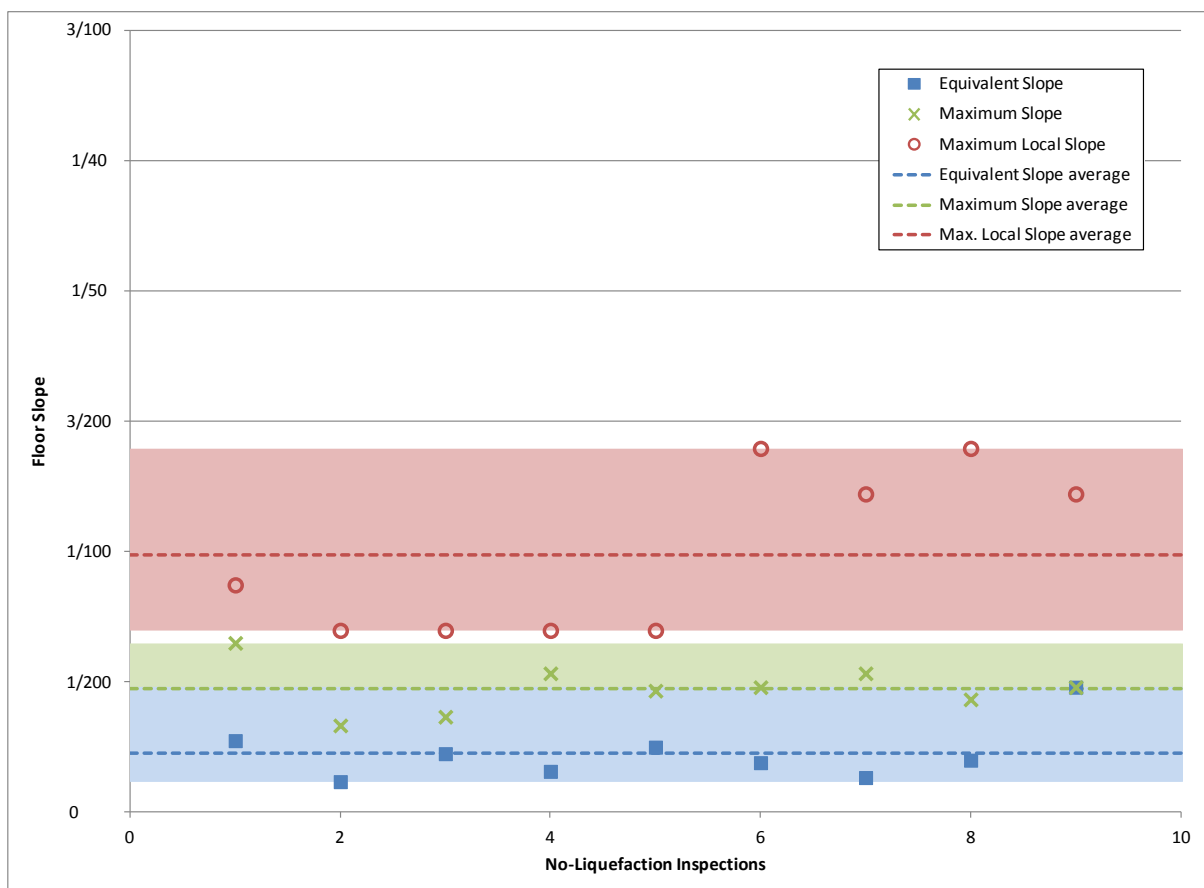


Figure 4.29. Concrete Perimeter foundation no-liquefaction inspections

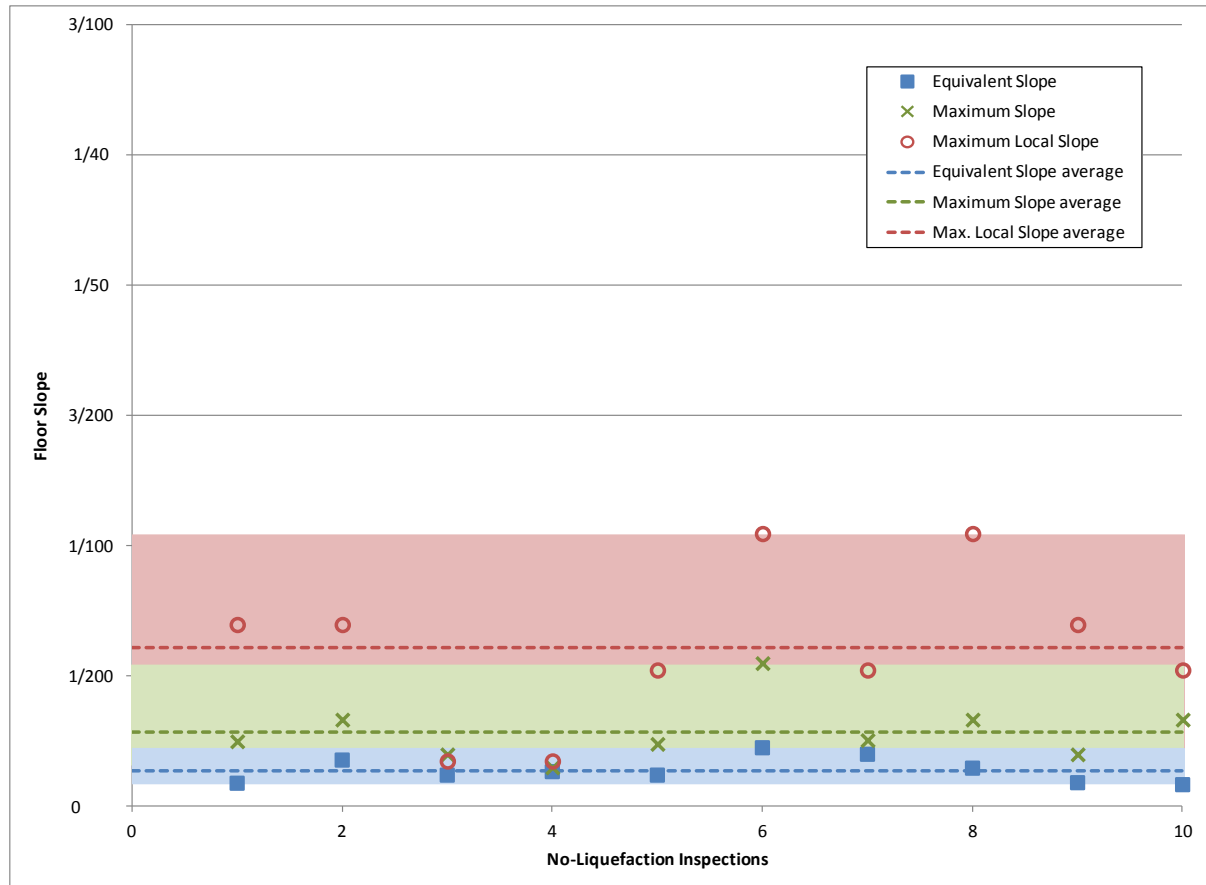


Figure 4.30. Slab-on-grade foundation no-liquefaction inspections.

When the average values for the three different slope indices for concrete perimeter and slab-on-grade foundations are compared to the other comparison values used, listed in Section 4.5.4, it is seen that the average equivalent slope for both concrete perimeter and slab-on-grade foundations, is close to the maximum overall slope measured by the DBH (2010) in new concrete slabs. The average maximum floor slopes for both foundation types are below the DBH (2011) criteria for a foundation that has suffered ‘no damage’, i.e. floor slope $< 1/200$ and differential settlement $< 50\text{mm}$ between any two points more than 2m apart. These findings confirm that foundations that did not suffer liquefaction performed satisfactorily, having close to no permanent deformation of their foundations beyond what is reasonably expected in a newly built floor, due to construction imperfections.

4.7.1.2 Superstructure Damage

Overall superstructure damage to the houses inspected in the non-liquefied areas was very low. For concrete perimeter, only two houses suffered moderate damage due to shaking, with the rest having low damage. For slab-on-grade houses the damage was even less, with six houses suffering effectively no superstructure damage due to the shaking in the earthquake alone and the other four with low damage. This supports the conclusions drawn by Buchanan & Newcombe (2010), Buchanan et. al. (2011) and Beattie et. al. (2011) that residential houses subjected to shaking alone in the Canterbury earthquakes performed well. A contributing factor to the favourable performance of the

houses not experiencing liquefaction may be the lower peak ground accelerations that were generally seen in the parts of Christchurch that did not suffer liquefaction (O'Rourke et. al., 2012).

4.7.2 Overall Performance

4.7.2.1 Ground Damage vs. Foundation Damage

For each of the four different foundation types, there is a clear trend that for an increased level of ground damage, foundation damage also increases. This supports the trend found in Chapter 3 with the 4th September 2010 earthquake data. This trend can be seen in Figure 4.31, which shows the median values for equivalent floor slope for each foundation type at each of the five possible levels of liquefaction severity recorded during inspections. It is also supported by Figure 4.32, which shows the median values for maximum floor slope for all foundation types. Both of these plots show that, in general, as the level of liquefaction increases, so does the floor slope in all foundation types, both over the whole floor, represented by the equivalent slope, and the maximum floor slopes.

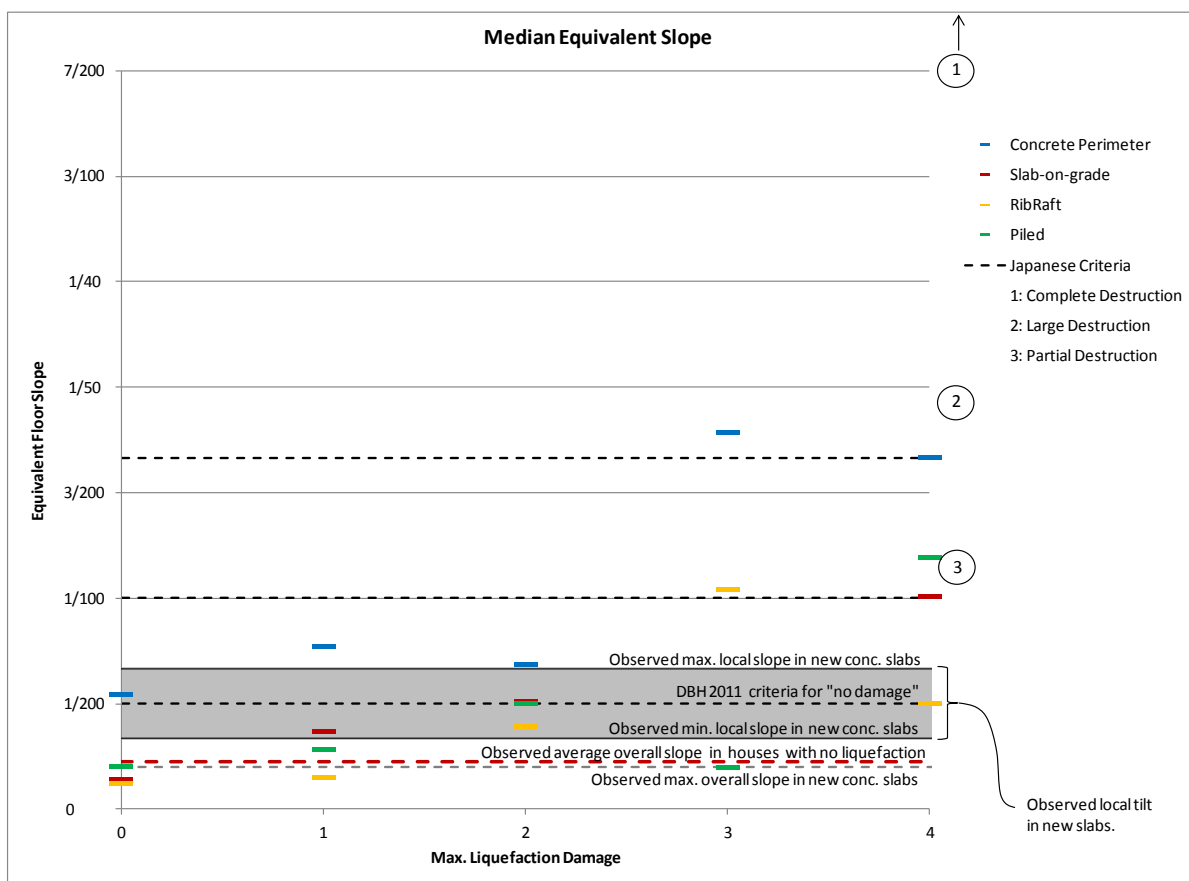


Figure 4.31. Median Equivalent Slope values at each level of liquefaction for each foundation type.

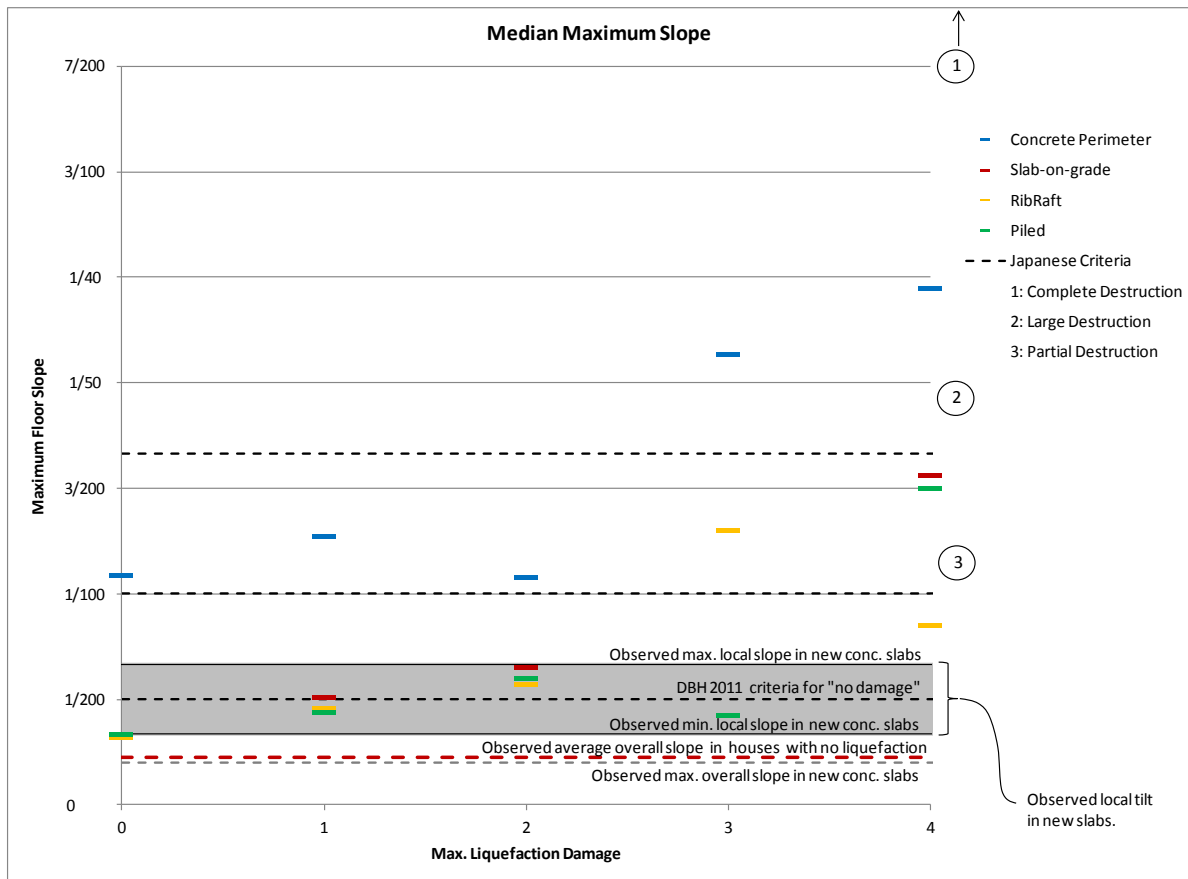


Figure 4.32. Median Maximum Slope values at each level of liquefaction for each foundation type.

4.7.2.2 Foundation Damage vs. Superstructure Damage

Figure 4.33 shows the distributions of superstructure damage for each level of foundation damage. It illustrates that for all buildings, as the foundation becomes more damaged, more houses suffer high levels of superstructure damage, and less houses have low superstructure damage. This agrees with the post-September data in Chapter 3. The proportion of houses in Figure 4.33 with no superstructure damage drops from 86% when houses also have no foundation damage, to less than 10% for houses with moderate or worse foundation damage, with all houses that had severely damaged foundations experiencing superstructure damage. The percentage of houses suffering high or severe superstructure damage in Figure 4.33 also increases as the foundation damage worsens. For houses with no foundation damage, only 14% had low superstructure damage, and none suffered worse than low damage. This supports the hypothesis from Chapter 3 that superstructure damage beyond the minor level is directly linked to foundation damage when there is liquefaction present.

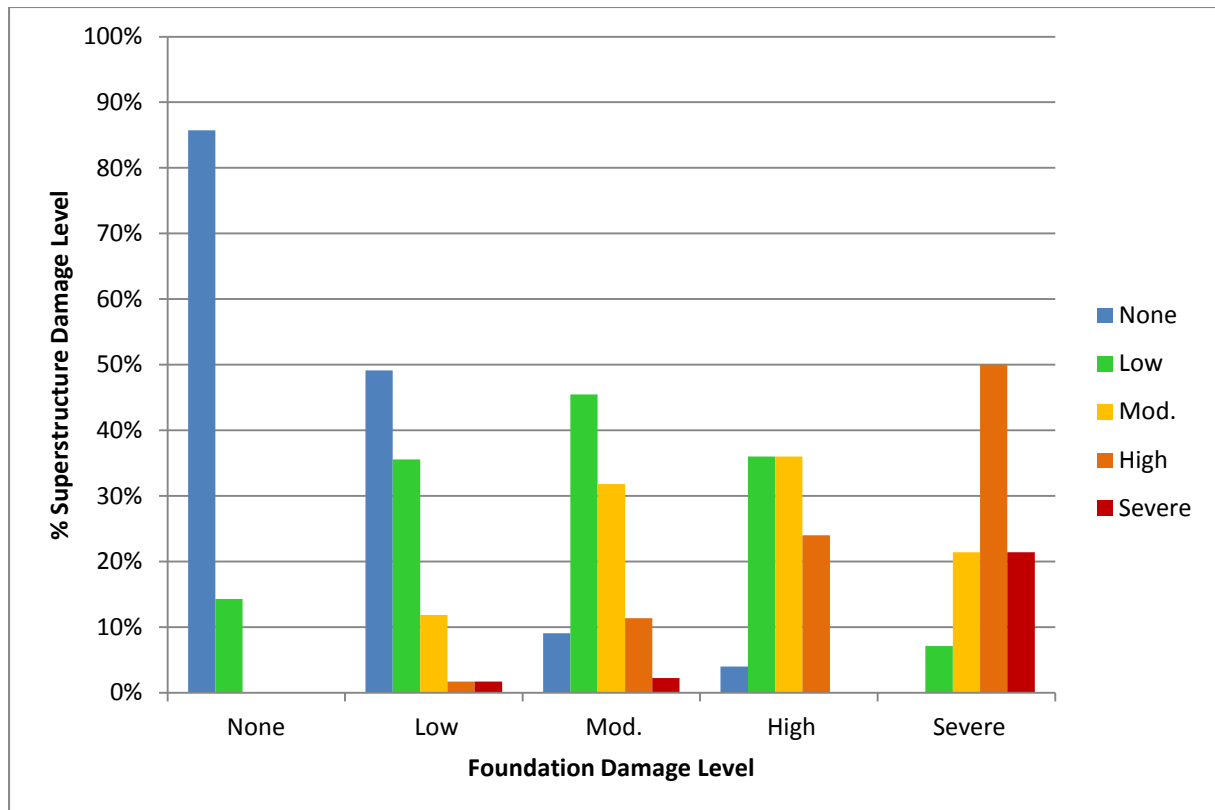


Figure 4.33. Distribution in percentage of superstructure damage level for differing levels of overall foundation damage.

4.7.3 Foundation Performance

4.7.3.1 Concrete Perimeter

Figure 4.34 shows the median values for the three floor slope indices at differing levels of liquefaction severity for concrete perimeter foundations. The relevant reference levels from Section 4.6.4 are also shown.

Figure 4.34 shows that, in general, the median floor slope increases for increasing liquefaction severity, agreeing with the general trend discussed in Section 4.7.2.1. The performance of concrete perimeter foundations when viewing Figure 4.34 is generally considered to be poor. The median values for the equivalent slope exceed the DBH (2011) criteria for a ‘non-damaged’ foundation of 1/200 at all levels of liquefaction, including where no surface ejecta was found. This suggests that concrete perimeter foundations subjected to any small degree of liquefaction loading will require some repairs.

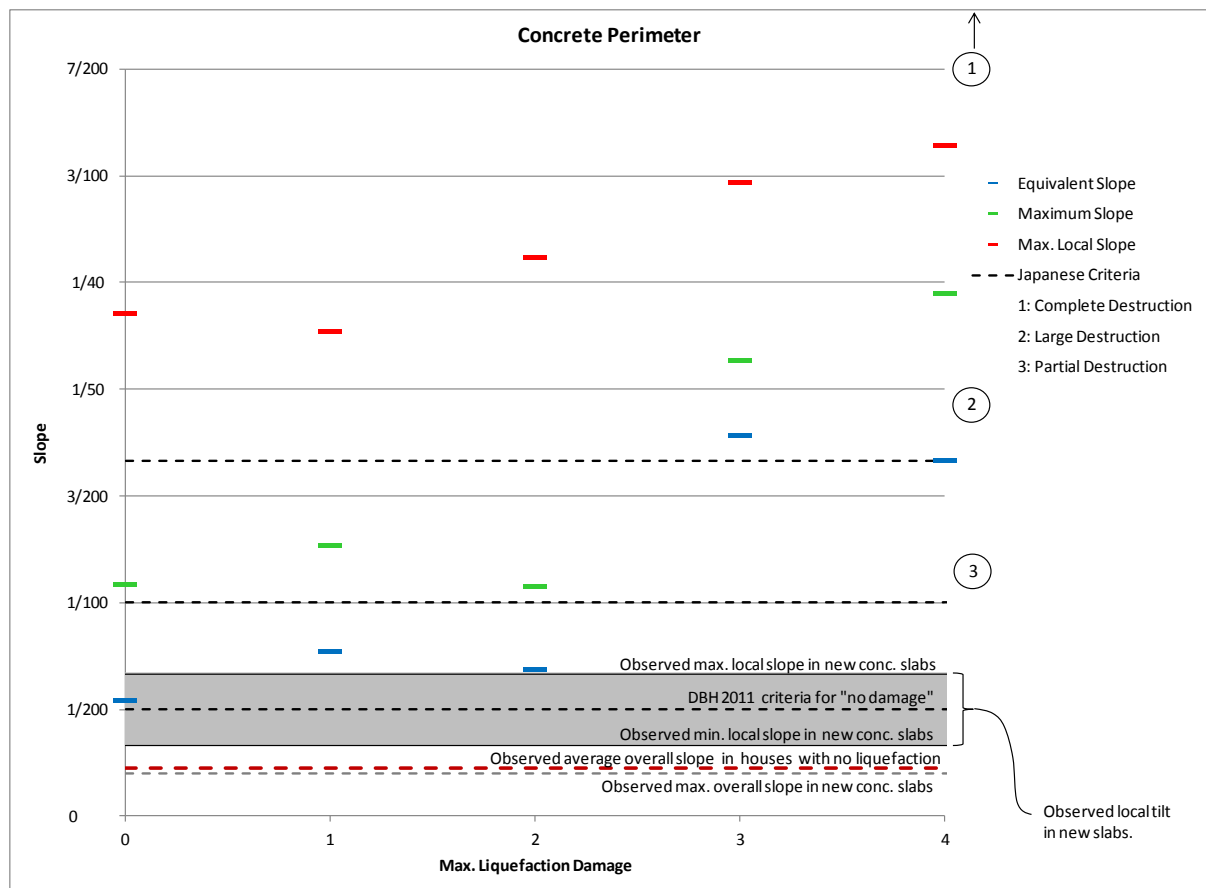


Figure 4.34. Median values for the three different floor slope parameters for each level of liquefaction severity for concrete perimeter foundations.

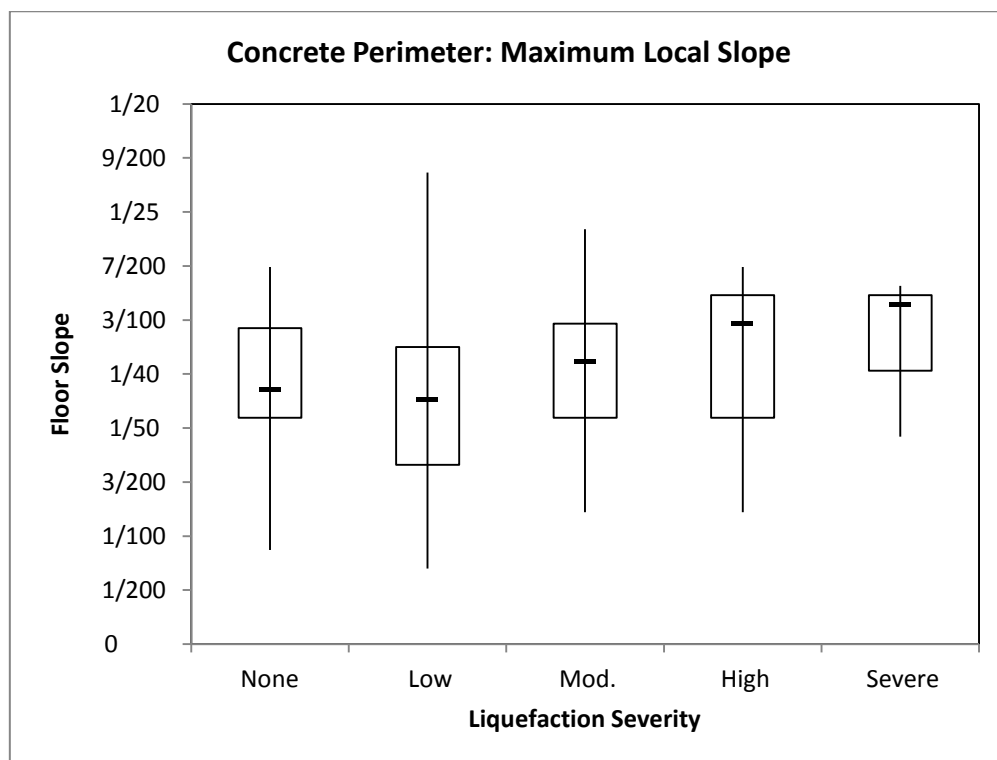


Figure 4.35. Box & Whisker plot for the maximum local slope of concrete perimeter foundations for each level of liquefaction severity.

The median maximum floor slopes are higher than the corresponding equivalent floor slope at every level of liquefaction, and are all above the Japanese criteria for partial destruction of 1/100. The median maximum local slopes at each level of liquefaction are even higher, and are all considerably higher than the Japanese criteria for large-scale destruction of 1/60. The median maximum local slopes are also consistently high, emphasised in the full box and whisker plot in Figure 4.35. The overall range of maximum local slopes recorded does not really increase with increasing liquefaction severity, unlike what is seen in the median values for maximum and equivalent floor slope. Even with no surface manifestation of liquefaction (i.e. 'none' in Figure 4.35), the median maximum local floor slope is almost 1/40, with the lowest local floor slope recorded close to the Japanese criteria for partial destruction of 1/100. For the no-liquefaction inspections in Figure 4.29, the average maximum local slope is at 1/100, much lower than the median values in Figure 4.35. This shows that once any small level of ground softening occurs, large localised slopes can occur in concrete perimeter floors. And if ground softening and liquefaction go beyond this, these local slopes are unlikely to increase very much. This is due to the flexible construction of this foundation type, which allows the floor humping deformation modes discussed in Section 4.5.3.

Figure 4.36 shows the raw data and median values for overall differential settlement at each severity of liquefaction damage. These are compared against the MBIE (2012) criteria for whether a foundation repair or rebuild is required. It can be seen that only a small number of concrete perimeter foundations inspected were below the threshold for requiring at least repair. Even with no surface manifestation of liquefaction, the median foundation will require repair, as was shown in Figure 4.34. Although no foundations inspected at this level required re-builds, foundations suffering low or moderate liquefaction are likely to need repair, and possibly rebuilding, with foundations suffering high or severe liquefaction almost always needing to be rebuilt. All foundations suffering significant lateral spreading, also shown in Figure 4.36, would require rebuilding. The two foundations with the highest differential settlements of 316mm and 474mm both suffered significant lateral spreading. This shows that significant lateral spreading is likely to cause significant foundation damage to concrete perimeter foundations.

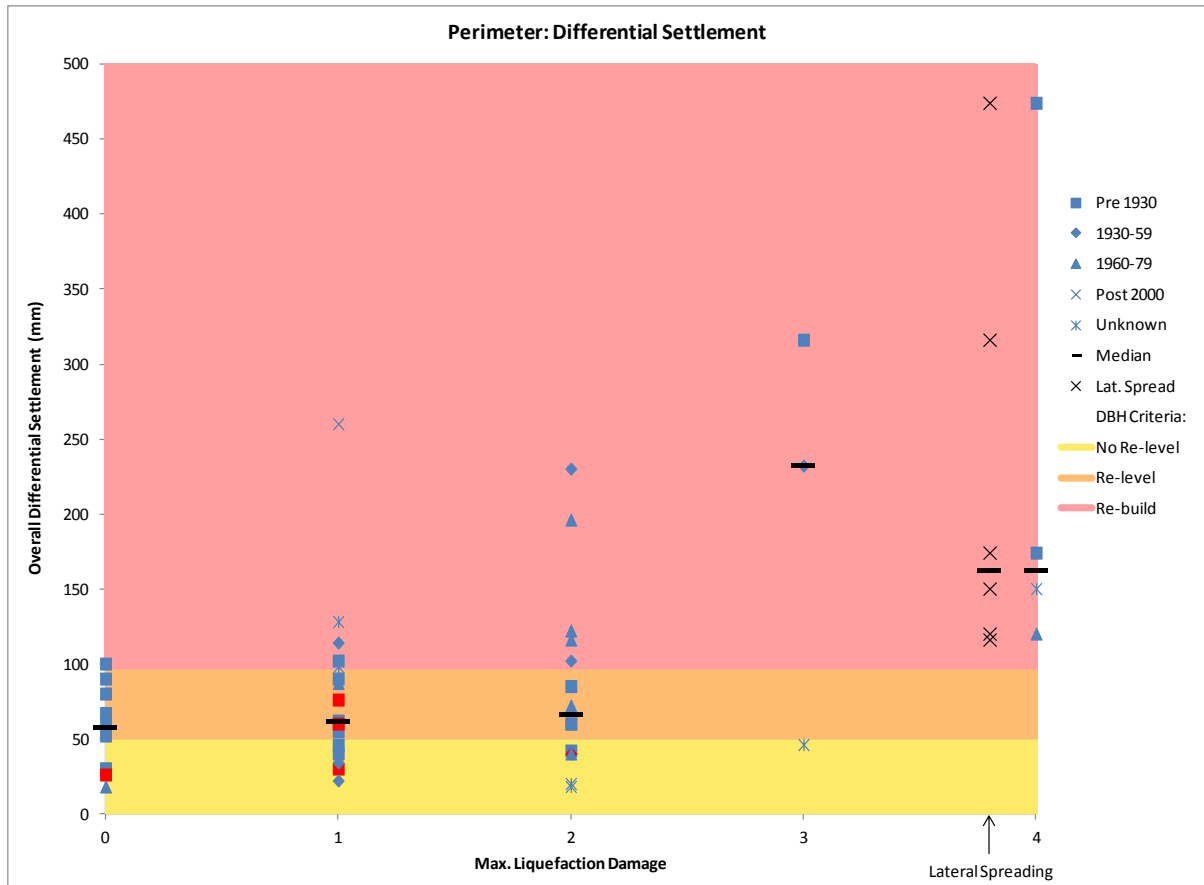


Figure 4.36. Overall Differential Settlement of concrete perimeter foundations against Maximum Liquefaction Damage, compared to the DBH (2011) criteria for repairing or rebuilding the foundation. Blue values are one-storey and red two-storey houses. Houses suffering significant lateral spreading are also identified.

Of the 49 concrete perimeter foundations inspected, more than half were built before the introduction of standards. This may also have influenced the performance of these concrete perimeter foundations. If old foundations do not perform as well, then it may not just be the type of foundation that is influencing the poor performance seen here. However, this data still represents the performance of a cross-section of the current concrete perimeter foundation building stock in Christchurch and New Zealand.

4.7.3.2 Slab-on-grade

Figure 4.37 shows the median values for the three different floor slope indices at each liquefaction severity level. It shows that for both no and low levels of liquefaction the foundations performed well, and considerably better than concrete perimeter foundations. The maximum local slope in both cases was close to or below the DBH (2011) maximum observed local slope in new concrete slab-on-grade floors of 1/150. The equivalent and maximum floor slopes for no and low liquefaction were also equal to or lower than the DBH (2011) criteria for a ‘non-damaged’ foundation of slope less than 1/200. The median equivalent slope for buildings suffering no surface manifestation of liquefaction was on the average line of 1/720 for the overall slope of houses that were inspected that suffered no liquefaction (i.e. those in Figure 4.13). For severe liquefaction the damage limits were exceeded, but this is to be

expected with the large loads under these ground conditions. These trends are also seen when the different floor slopes are plotted against ground settlement from the LiDAR data, shown in Appendix N.2.

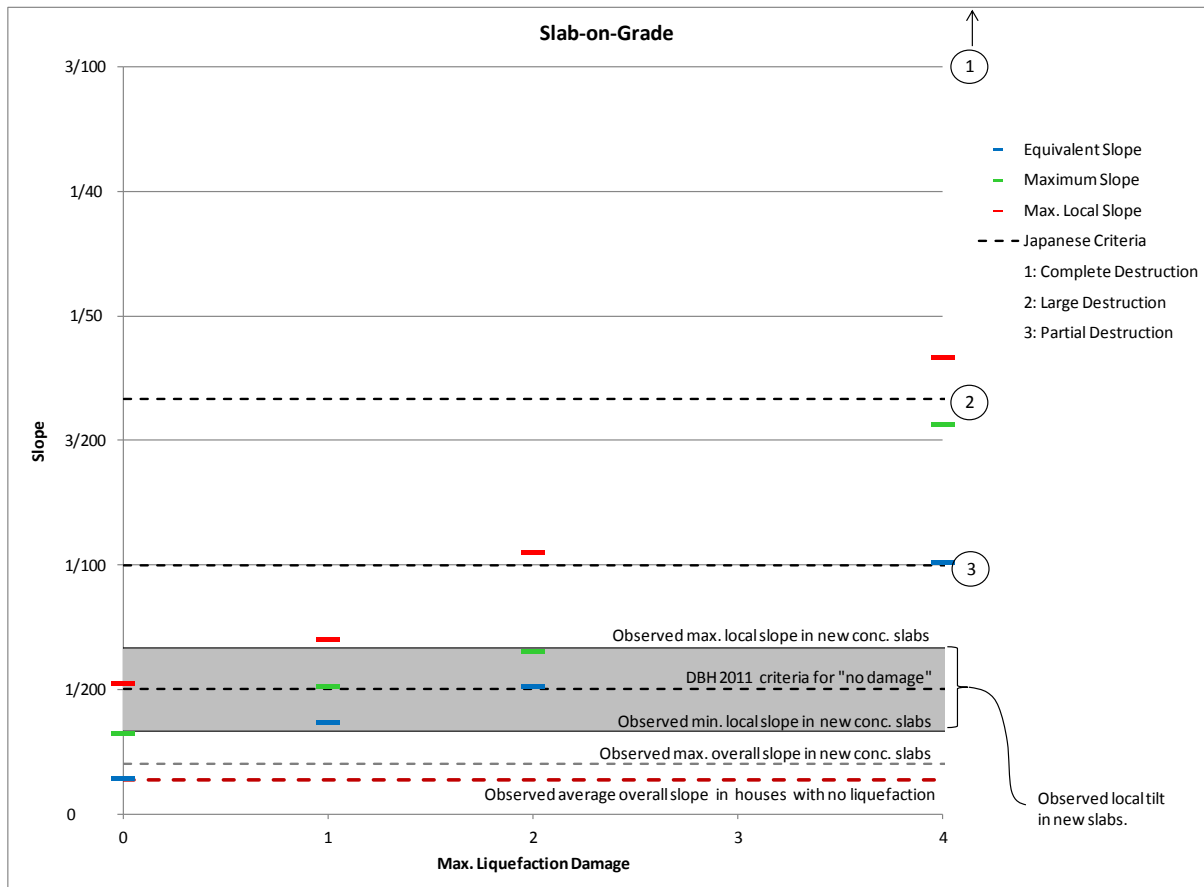


Figure 4.37. Median values for the three different floor slope parameters for each level of liquefaction severity for slab-on-grade foundations.

Unlike the data for concrete perimeter foundations, all three slope indices increase at close to the same rate as the liquefaction severity increases. This is to be expected, as slab foundations are more cohesive and less flexible than the timber floors of a concrete perimeter foundation. At low levels, their deformation is more gradual and floor slopes are more even, with less localised deformations. And at high levels of deformation, broken tilting often occurs, with the broken sections tilting uniformly within themselves. This means that at any level of deformation, the three different slope indices are more likely to be closer together than for concrete perimeter foundations where localised floor humping occurs.

Figure 4.38 contains the slab-on-grade foundation data for differential settlement at each level of liquefaction. It also shows that slab-on-grade foundations performed considerably better than concrete perimeter foundations. There were no foundations with differential settlement above 50mm (i.e. requiring repairs) when no liquefaction was seen on the ground surface, and at low levels of liquefaction, only three of the buildings inspected required repair. The median foundation suffering

moderate liquefaction would need repairing with a differential settlement of 53mm, but at this level of liquefaction there is still a good chance that no repair is required. The lack of data for slabs suffering high and severe liquefaction prevents adequate conclusions from being drawn at those levels, but of all the slab foundations inspected, only a small number needed rebuilding.

Figure 4.38 also shows the differential settlements of the houses which suffered significant lateral spreading. It is clear from these data points that lateral spreading can cause significant deformation in slab-on-grade houses, with only two houses below the no re-level threshold of 50mm differential settlement. Two houses also require rebuilding with differential settlements of 166mm and 202mm.

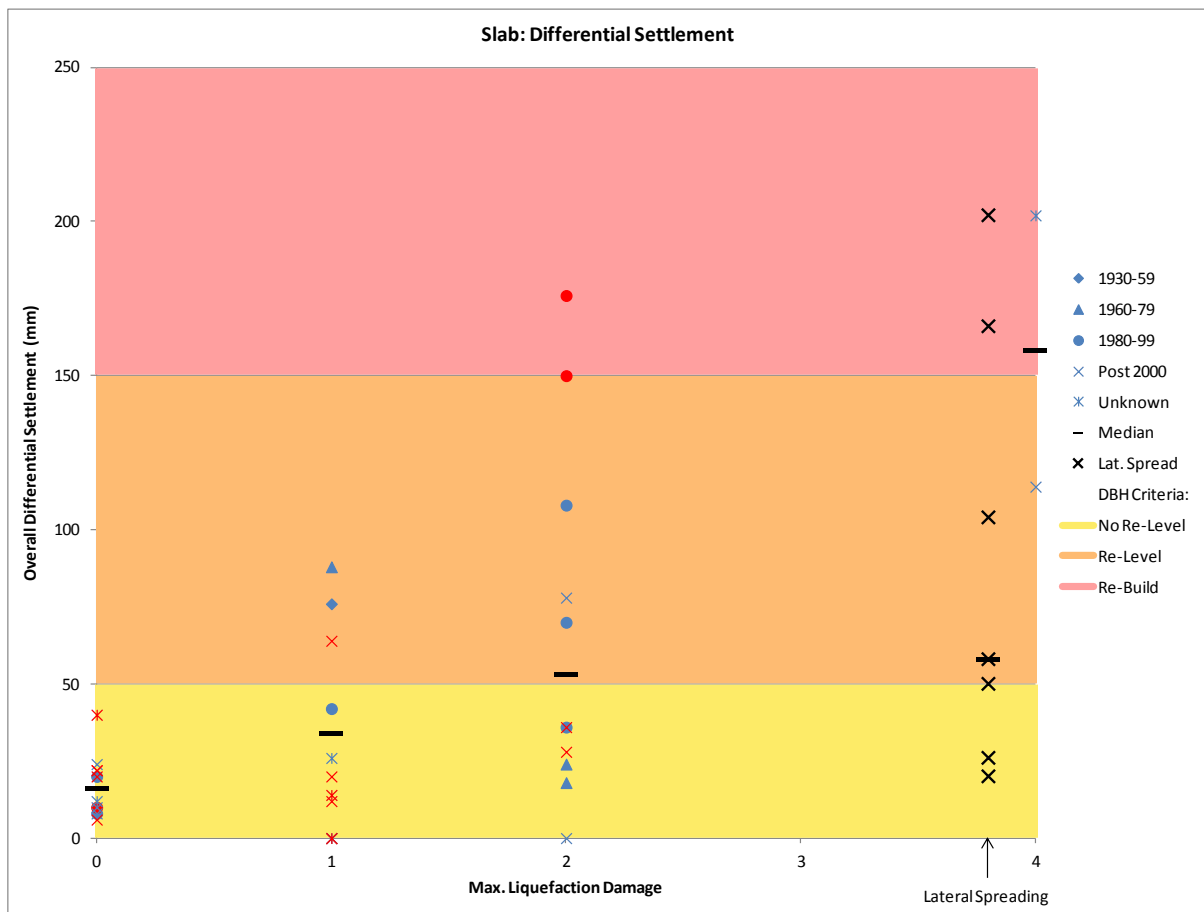


Figure 4.38. Overall Differential Settlement of slab-on-grade foundations against Maximum Liquefaction Damage, compared to the DBH (2011) criteria for repairing or rebuilding the foundation. Blue values are one-storey and red two-storey houses. Houses suffering significant lateral spreading are also identified.

4.7.3.3 RibRaft

Figure 4.39 shows the median values for the three different floor slope parameters at each level of liquefaction severity for ribraft floors. For liquefaction up to moderate severity, the performance of ribraft foundations was better than for slab-on-grade foundations. The median values for maximum local slope were close to or below the maximum local slope in new concrete floors of 1/150 as found by the DBH (2011). The equivalent and maximum floor slopes for liquefaction up to moderate severity were all close to or below the DBH (2011) criteria for a ‘non-damaged’ foundation of a 1/200

slope, and the median equivalent slope for both no and low liquefaction levels was equal to the average overall floor slope in foundations inspected that didn't suffer liquefaction of 1/720. At high and severe levels of liquefaction the median values for all three floor slopes had higher values, as for the slab-on-grade foundations, again suggesting significant damage under the larger foundation loads. These trends are also found when the floor slope data is graphed against LiDAR ground settlement, the results of which are shown in Appendix N.2.

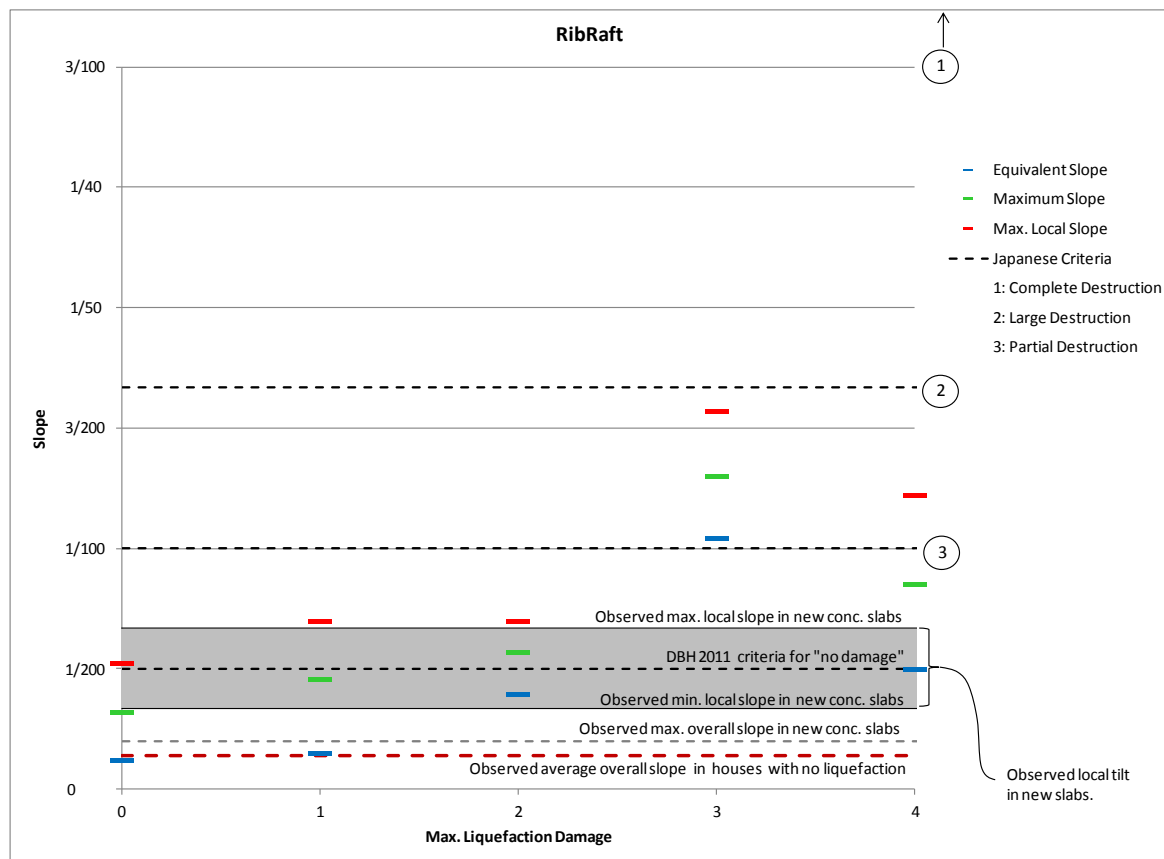


Figure 4.39. Median values for the three different floor slope parameters for each level of liquefaction severity for ribraft foundations.

An improvement in performance between ribraft and slab-on-grade foundations is also seen in Figure 4.40 which shows all the ribraft data for differential settlement against liquefaction severity. For none and low liquefaction levels, there were no foundations with differential settlements higher than 50mm and therefore requiring repair. For moderate or high liquefaction, no more than two foundations required re-levels with differential settlements over 50mm. There was one ribraft foundation inspected that required a foundation re-build, but this was considered in the liquefaction analysis as the house suffered significant lateral spreading. As for slab-on-grade foundations, more data is required of foundations suffering high and severe liquefaction severity to draw any definitive conclusions as to whether ribraft foundations required re-building.

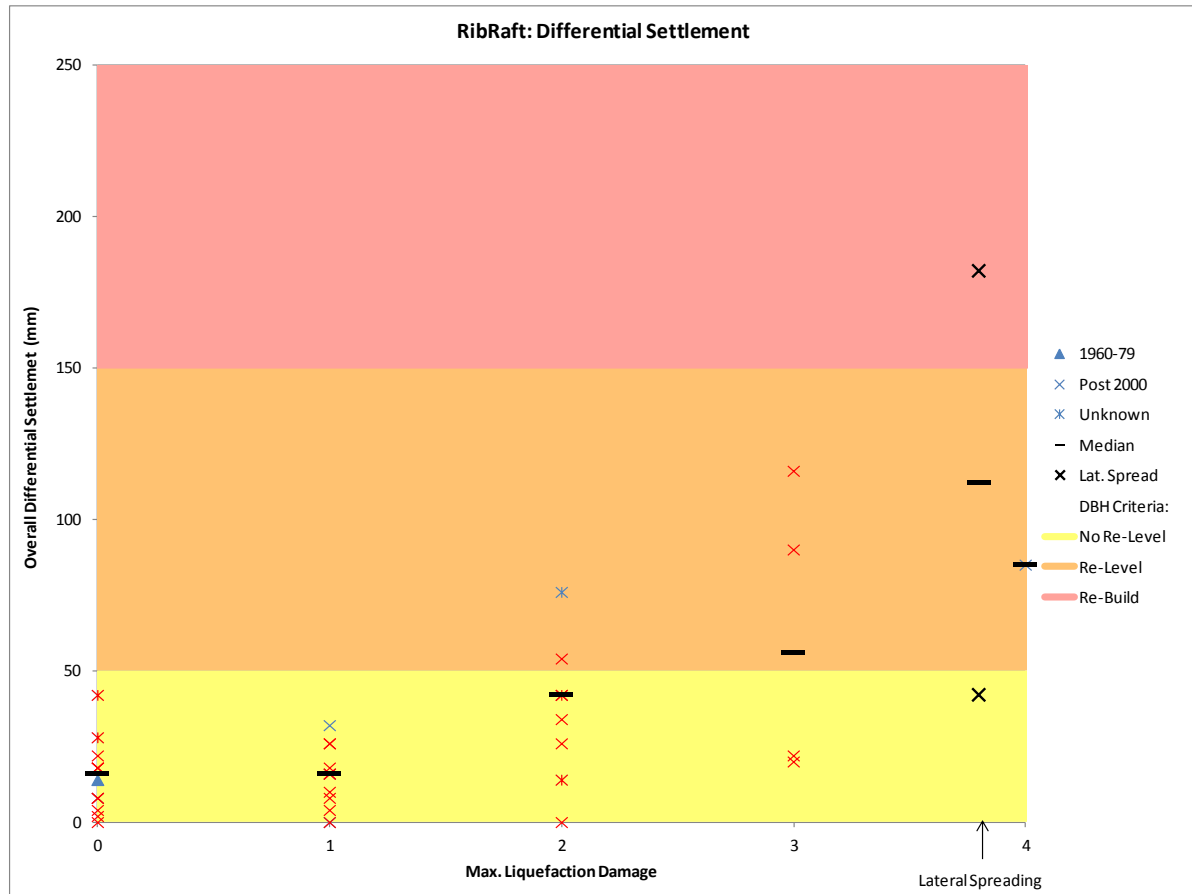


Figure 4.40. Overall Differential Settlement of RibRaft foundations against Maximum Liquefaction Damage, compared to the DBH (2011) criteria for repairing or rebuilding the foundation. Blue values are one-storey and red two-storey houses. Houses suffering significant lateral spreading are also shown.

These trends suggest that ribraft foundations are better at resisting break-up and differential tilting under liquefaction-related loading, due to their higher strength and stiffer construction. They deform more uniformly than standard slab-on-grade foundations. This is particularly evident for the data up to and including moderate liquefaction, where there are significantly less ribraft foundations requiring repairs than for slab-on-grade foundations.

The two ribraft houses inspected that suffered significant lateral spreading, shown in Figure 4.40, suffered very different damage levels. One had a differential settlement of 182mm, requiring rebuilding while the other didn't even need repair, with a differential settlement of 42mm. The one badly damaged house does show that despite the extra strength and stiffness in ribraft foundations, under large imposed deformation and loading they can still be severely damaged.

4.7.3.4 Piled

Figure 4.41 shows the median values for all three floor slope indices at the different levels of liquefaction for piled foundations. A lack of data at high and severe liquefaction levels again prevents conclusions being drawn about behaviour under the more extreme ground conditions, but for liquefaction up to moderate severity the performance of the piled foundations is very similar to ribraft foundations, and again better than slab-on-grade foundations. For liquefaction up to moderate severity

the median maximum local slopes were all below the maximum local slope in new concrete slabs of 1/150 from the DBH (2011), showing slightly better performance for local floor deformations than ribraft foundations. The median equivalent and maximum floor slopes are similar and slightly higher than for ribraft foundations, but again all are close to or below the DBH (2011) criteria for a 'non-damaged' foundation.

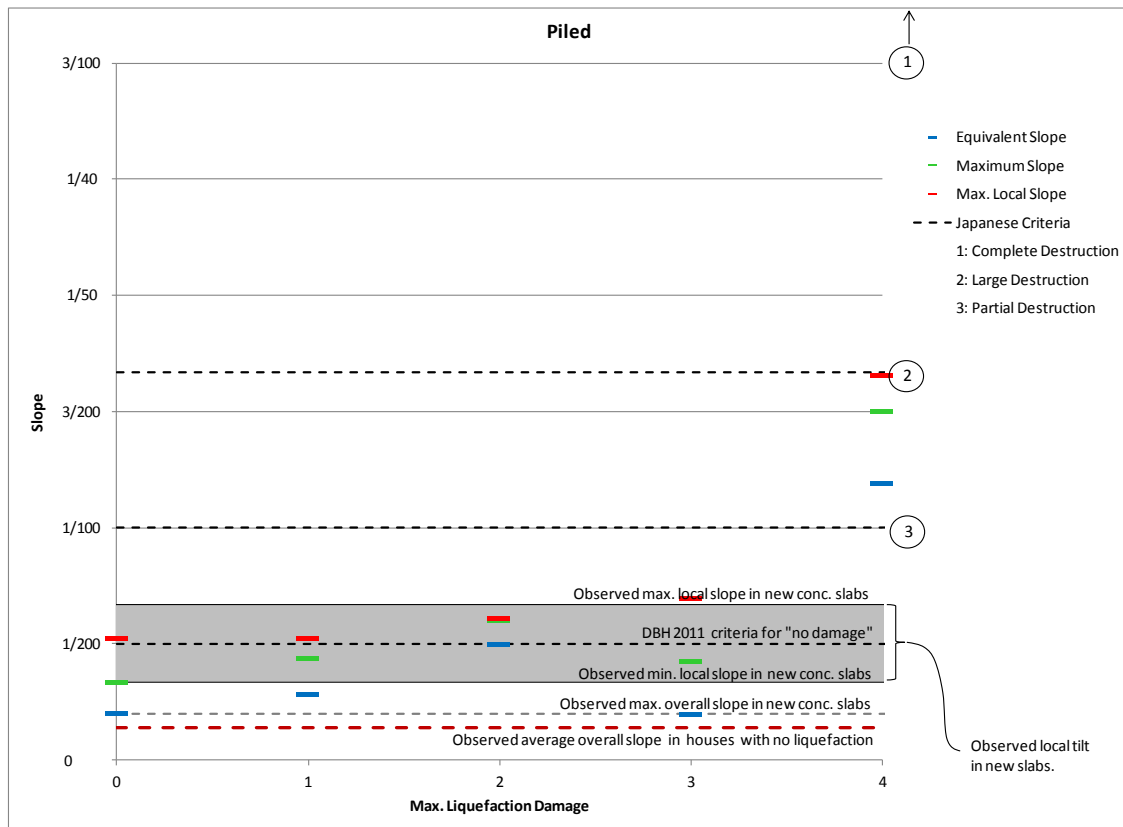


Figure 4.41. Median values for the three different floor slope parameters for each level of liquefaction severity for piled foundations.

Figure 4.42 shows the differential settlement data for piled foundations at each level of liquefaction severity. It also shows that the performance of piled foundations is better than for slab-on-grade foundations. There are only three piled foundations with differential settlement more than 50mm, and so requiring repair, at low levels of liquefaction. This is the same as for slab-on-grade foundations, but there are also only three foundations requiring repair at moderate levels of liquefaction, with no houses inspected that required a re-build (based on the MBIE (2012) criteria).

There is again a lack of data for high and severe liquefaction, preventing discussion around high land damage, however, like the ribraft slabs, it seems unlikely that many foundations would need re-building. Figure 4.42 also shows the differential settlement of the three piled foundation houses which suffered significant lateral spreading. Two of the houses performed well under the extreme loading, experiencing 22mm and 64mm of differential settlement, with the second one requiring a re-level. However the third house had a differential settlement in excess of 250mm, requiring a foundation re-build. The superstructure of this third house performed very well, as the house was tilting uniformly,

putting little stress on the superstructure. However, due to the deep liquefaction and lateral spreading, a re-build of the foundation would be required.

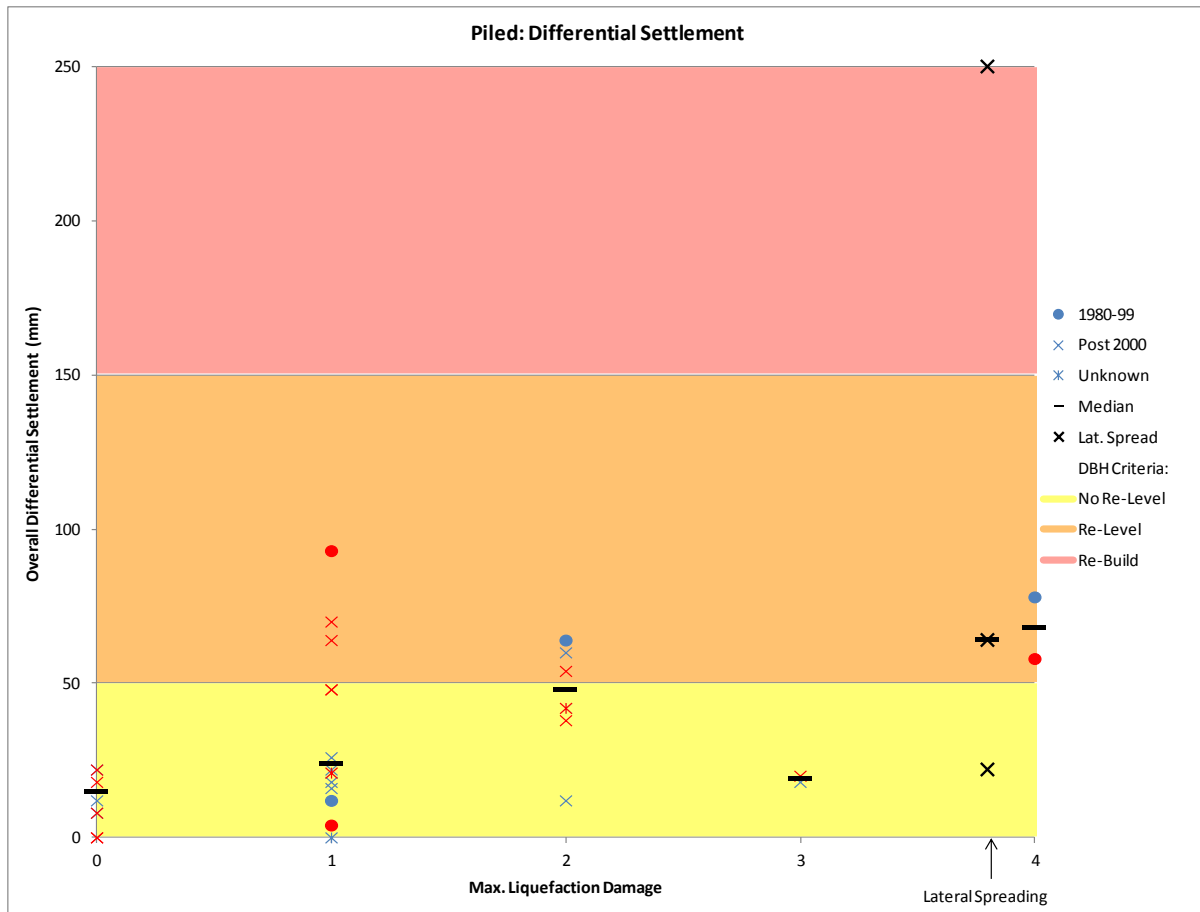


Figure 4.42. Overall Differential Settlement of Piled foundations against Maximum Liquefaction Damage, compared to the DBH (2011) criteria for repairing or rebuilding the foundation. Blue values are one-storey and red two-storey houses. Houses suffering significant lateral spreading are also shown.

4.7.3.5 Overall vs. Local Floor Slopes

When comparing the three different floor slope indices, in Figure 4.34, Figure 4.37, Figure 4.39 and Figure 4.41 for concrete perimeter, slab-on-grade, ribraft and piled foundations respectively, a clear trend is apparent. For all foundation types, at all levels of liquefaction, higher slope values are more likely the more localised the measurement is. This is also seen when the data is plotted against the ground settlement from LiDAR data, shown in Appendix N.2. When looking at the nature of how each index is calculated in Figure 4.28, this is to be expected. Aside from the case of a perfectly stiff foundation, that only ever tilts and settles uniformly as a rigid body, whenever any foundation deforms, higher slopes will be found locally.

The only difference in performance is when concrete perimeter foundations are compared with the slab-based foundations. Concrete perimeter foundations have far higher local slopes in relation to their overall slopes than slab-based foundations. This is due to their greater flexibility, which causes their unique deformation mode, as discussed in Sections 4.4.1.9, 4.4.1.10 and 4.6.3.1.

4.7.3.6 Concrete Perimeter vs. Slab-Based Foundation Types

The perimeter foundation houses behaved quite differently to the three slab-based foundation types, both in their deformation mode and level of performance.

Perimeter foundations were much more easily damaged by the effects of liquefaction than the slab-type houses. This can be both a negative and positive. The negative being that for low levels of liquefaction, there is likely to be significant damage to perimeter foundations, requiring repair or even re-build, whereas for the slab-based foundations, re-build is very unlikely, and major structural repairs may not even be needed, rather just cosmetic touch-ups to the superstructure.

However, once the liquefaction level increases enough that both perimeter and slab-based foundations are seriously damaged (i.e. beyond moderate levels of liquefaction) having a perimeter foundation can be an advantage. So long as the superstructure is not irreparably damaged (which is more likely in a house with a perimeter foundation, as the floor is more flexible, and not as fixed to the superstructure), the house can be (relatively) easily lifted up and the foundation replaced. For slab-based houses though, once a certain level of damage is reached, re-laying of the foundation becomes quite difficult, as the superstructure is attached directly to the foundation.

The deformation mode for perimeter foundations is characterised by cracking in the perimeter, which can be anything from hairline to more than 100mm in lateral spreading situations. This is accompanied by humping in the middle of floors, either over the whole footprint of the house, or locally, room by room. This humping is caused either by the settlement of the perimeter footing, which supports the outer walls and roof, or by uplift of the piles under the floor by liquefaction ejecta, or a combination of both. These characteristic features are then added to by overall tilting and other effects in more seriously damaged foundations. The slab-based foundations have a different mode of deformation, characterised by more subtle and consistent floor sloping, often based on different sections of the slab. In more severely damaged foundations, the slab can break up, giving different floor slopes throughout the floor plan.

As shown in Figure 4.37, Figure 4.39 and Figure 4.41, the three slab-based foundations behaved similarly, at least up to moderate liquefaction levels where there was sufficient data for robust comparisons. All had similar performance in all three floor slope indices, with median maximum local floor slopes generally close to or below the maximum local slopes found in new concrete slabs by the DBH (2011) of 1/150, and all had median equivalent slopes at or below the DBH (2011) criteria for a 'non-damaged' foundation of 1/200 up to moderate levels of liquefaction. These values are directly compared in Figure 4.43, which shows the median values for each of the three different floor slopes (Equivalent Slope, Maximum Slope and Maximum Local Slope) for each foundation type for no surface liquefaction. The three slab-based foundations have very similar values for all three slope

indices, and this trend is generally repeated for low and moderate levels of liquefaction, shown in Appendix N.1.5 and Figure 4.44. The perimeter foundations however have much larger values for all three floor slope types, with the difference increasing for the more localised slope measurements. This confirms that perimeter foundations are more susceptible to damage, and illustrates the effect of the more flexible damage mode of perimeter foundations, with the high maximum local slope caused by local humping of the interior floors.

For moderate liquefaction, shown in Figure 4.44, the median floor slopes in concrete perimeter foundations are still considerably higher than those for the three slab-based foundation types. Although the difference is somewhat less than for no liquefaction, it supports the idea that concrete perimeter foundations are damaged more easily than slab-type foundations, and this damage results in higher floor slopes.

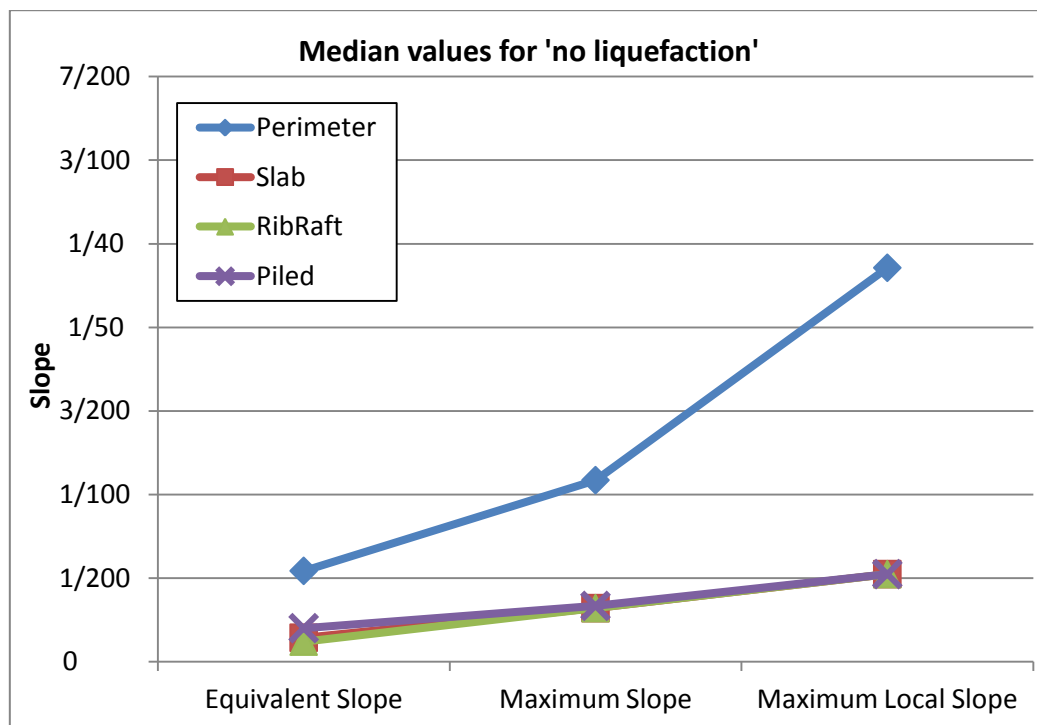


Figure 4.43. Median slope values for each foundation type, for 'no liquefaction', for the three floor slope indices.

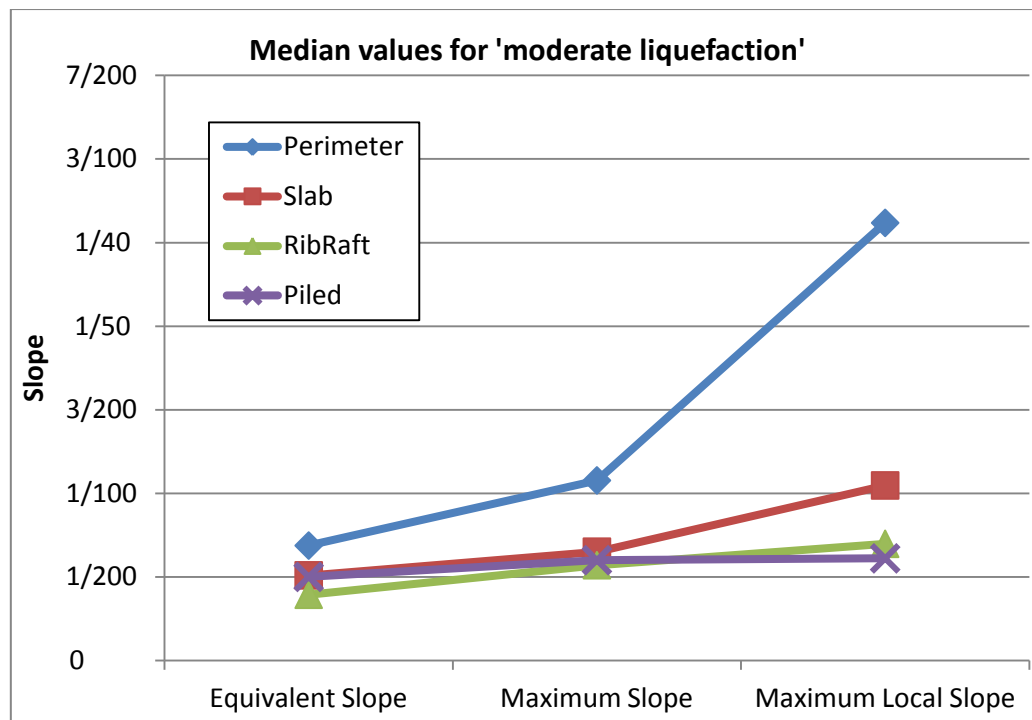


Figure 4.44. Median slope values for each foundation, for 'moderate liquefaction', for the three floor slope indices.

4.7.3.7 Slab-on-grade vs. RibRaft/Piled Foundations

As already stated, Figure 4.43 and the additional plots in Appendix N.1.5, show that at low levels of liquefaction, the three slab-based foundation types have very similar median values for all three slope measurements, although for moderate liquefaction slab-on-grade foundations have a higher median maximum local slope than ribraft or piled foundations. This is because at low levels of liquefaction, the loads on the foundation are not high enough to break up any of these three foundation types, so they all deform in a similar way, creating comparable distributions of floor slope. At moderate liquefaction, the loads are getting high enough that the weaker slab-on-grade foundations are beginning to break up causing higher floor slopes.

Under higher liquefaction levels the performance of the three slab-based foundations is somewhat different. Although there was insufficient data to examine the performance of the three foundation types at high and severe liquefaction levels, even under moderate liquefaction, shown in Figure 4.44, slab-on-grade foundations performed worse than ribraft and piled foundations, particularly in the local sense. This is due to their weaker and relatively more flexible slab that will begin to break up under lower loads. Figure 4.40 and Figure 4.42 show that ribraft and piled foundations both can be expected to require structural repairs due to the higher loads on the foundation at high and severe liquefaction levels, but it is clear that ribraft and piled foundations performed better than slab-on-grade foundations, as at moderate levels of liquefaction there were already significant numbers of slab-on-grade foundations requiring repairs, with two slab-on-grade foundations shown needing rebuilds, one of them with only moderate liquefaction (Figure 4.38). There are no ribraft or piled foundations in these figures that needed rebuilds.

This data shows that if ground softening or very low levels of liquefaction are possible, it is acceptable to use slab-on-grade foundations as these stand up to the loading just as well as the more specialised ribraft and piled foundations. However, once liquefaction exceeds these low levels, it is better to use the specialised foundations with more stiffness and strength. They are likely to suffer lower damage and are less likely to require rebuilding, with a higher possibility of no repairs being required at all. The small differences in the median values for the local and overall floor slopes are more likely due to the specific dataset collected, and it can be generally said that ribraft and piled foundations performed very similarly, and better than slab-on-grade foundations.

It is not recommended to use a concrete perimeter foundation with details based on New Zealand practice, as once any significant levels of ground softening occur these foundations perform considerably worse than slab-based foundations causing high floor slopes often requiring repairs or rebuilding.

4.7.4 Superstructure Performance

4.7.4.1 Cladding Performance

When all the properties, regardless of foundation type, were separated into the external cladding types ‘light’ (weatherboard, iron, stucco and plywood) and ‘heavy’ (brick, brick veneer, stone veneer, concrete slab, concrete block and concrete block veneer) there was very little difference in performance between the two types. When superstructure damage against foundation damage was plotted, there was no discernible difference in the performance of lightweight or heavyweight claddings. This was also the same when the various foundation damage indices were plotted for the different liquefaction severities. The cladding weight appears not to have affected foundation performance.

4.7.4.2 Single-Storey vs. Two-Storey Houses

The data in Figure 4.36, Figure 4.38, Figure 4.40 and Figure 4.42 is graphed by age and by one-storey (blue data points) and two-storey (red data points) houses. When reviewing these figures, there is no obvious difference between the two data sets. All construction periods for all foundation types had buildings suffering higher and lower damage than the median at each level of liquefaction. For some foundation types there was limited data for one or two-storey houses, but overall the data suggests that number of storeys does not affect the performance of foundations or the structure as a whole.

4.8 Discussion

4.8.1 Overall Performance

General trends were found in the data to support the findings in Chapter 3 with the inspection data collected following the 4th September 2010 earthquake. For all foundation types, as the severity of ground damage increases, whether it be liquefaction rating or ground settlement, so too does the damage to the foundation. This continues to the superstructure, in that for increasing foundation damage, the damage to the superstructure is more likely to be worse.

For the houses inspected in the West of Christchurch which did not suffer any liquefaction, the overall performance was very good. This is to be expected, as the superstructures of timber-framed residential houses in NZ are very resilient to inertial shaking from earthquakes, as they are flexible, with a large degree of redundancy and numerous load-bearing and dissipating mechanisms. This agrees with the findings of Buchanan & Newcombe (2010), Buchanan et. al. (2011) and Beattie et. al. (2011). In general, only minor cosmetic damage to the superstructure was caused by inertial loading. In contrast, loading situations caused by liquefaction are often displacement-based, causing uplift or sinking of all or part of the foundation, or even stretching in cases of lateral spreading. These permanent displacements put large loads on the structure, and cause it to distort in ways it is not meant to. This is the reason for the trends in the data of increasing foundation damage with land damage. These loads and foundation deformations can cause large levels of cracking to interior and exterior cladding, and pulling-out of nails in the framing. This is why foundation damage is so closely related to superstructure damage, as the foundation is damaged by displacements, which then force the superstructure through the same movements.

4.8.2 Foundation Performance

Concrete perimeter foundations were generally found to have performed poorly when subjected to the effects of liquefaction, contrary to the findings of Buchanan et. al. (2011). This is most likely due to the weak, flexible nature of the foundations, where small piers support the internal floor without rigid connections, and are not connected to the concrete perimeter wall supporting the walls and roof. Sometimes in older houses even the floor and sub-floor structure are not connected rigidly to the concrete perimeter.

The flexible timber and lack of a cohesive foundation system means these foundations deform under little load, and lead to the typical deformation modes seen in concrete perimeter houses of room and overall floor humping (discussed in Sections 4.4.1.9 and 4.4.1.10 respectively). These deformation modes lead to the very high maximum local slopes seen in Appendix N.1.1, even at low levels of settlement and liquefaction, supporting the idea that concrete perimeter foundations are damaged

easily, at least in the local sense. However, these high slopes do not increase despite the increasing ground settlement.

The slab-based foundations performed better than concrete perimeter foundations with lower floor slopes for all indices throughout the range of land damage severities, and much closer values between the three slope indices. This is due to their stiff, cohesive structural system, compared to the weak, flexible and component-based concrete perimeter system. Slab foundations deform more gradually, leading to localised slopes closer in value to the corresponding overall deformation measures.

The two specialised foundation types, ribraft and piled, performed the best. For ribraft foundations, this is due to their increased stiffness and strength, which allows them to resist larger loads from the ground before deforming significantly and breaking up. Their structural form does not prevent overall global or differential settlement of the structure, but reduces deformations within the foundation. For piled foundations, the increased support and reduced loading area provided by the piles reduces the actions that must be resisted by the slab floor. This means reduced deformations and an increased ability to resist global and differential settlements over standard slab-on-grade foundations.

From the small dataset of houses which experienced significant lateral spreading, it was shown that lateral spreading can cause severe differential settlement up to 250mm and result in significant structural damage in all types of foundations, due to the extreme loading conditions caused by his type of ground deformation.

Although the data is inconclusive, it suggests that building weight, either cladding type or number of storeys, does not affect the level of superstructure damage or increase settlement or foundation damage. This supports the data from Chapter 3, and will be investigated further in Chapter 6.

4.8.3 Sampling Issues

The main issue encountered during this research process was the difficulties in obtaining a sufficient spread of houses both in foundation and structural type and across the different levels of liquefaction severity. This was caused by a number of factors.

First was the large local variability in ground damage levels within the wider identified areas of land damage on the UC liquefaction map. Although there is a large residential ‘red zone’ (New Zealand Government, 2011) based on land damage, within this, individual properties could have experienced land damage anywhere on the scales shown in Table 4.4. This is due to the local variability in soil profiles and ground conditions within Christchurch, and made it difficult to achieve the desired spread of land damage severities.

This problem was compounded by the small number of ribraft and piled foundations in Christchurch. Because of this, and the need to obtain permission for the inspections, which was not always given, it

was not possible to be selective with which houses were inspected of these foundation types to ensure an even spread of data across all land damage categories and house types. As it was also necessary to inspect all foundation types close to each other where possible, as discussed in Section 4.3.1.1, this meant that the spread of concrete perimeter and slab-on-grade houses was also not ideal, although they were better than ribraft and piled due to some double-ups. These factors caused the spread of inspected houses to be weighted towards areas which suffered lower levels of liquefaction damage, with only a few houses of each foundation type representing the high to severe land damage categories. This limited the ability to draw conclusions on the performance of the different foundation types at higher levels of ground damage. The range and number of house types inspected also made it difficult to draw conclusions on the influence of various construction factors such as cladding weight and number of storeys on performance.

Despite these drawbacks, the data set gathered for this research provides a detailed picture of the performance of houses and different foundation types subjected to liquefaction in the Canterbury earthquakes, and allows some valuable conclusions to be drawn.

4.8.4 Summary of Key Findings

There were a number of key trends identified in the performance of residential houses and their foundations when subjected to a full range of liquefaction effects, as occurred in the 22nd February 2011 earthquake. These findings are summarised here:

- The performance of the houses inspected that suffered no liquefaction was good, agreeing with the findings of Buchanan & Newcombe (2010), Buchanan et. al. (2011) and Beattie et. al. (2011).
- For all foundation types, as the severity of liquefaction increased, so too did the damage to the foundation. This agrees with the findings in Chapter 3.
- With increasing foundation damage, the damage to superstructures also increased, also supporting the findings in Chapter 3.
- Concrete perimeter foundations performed poorly. Based on median values, concrete perimeter foundations required at least repair at all levels of liquefaction, both for equivalent floor slope and differential settlement, with all median values above the DBH criteria for a non-damaged foundation of 1/200 floor slope or 50mm differential settlement.
- Concrete perimeter foundations had very high and consistent maximum local floor slopes, all above 1/50, regardless of liquefaction severity. This was due to the weak and flexible foundation system allowing local humping in the floor, producing large local slopes from very low liquefaction levels.
- Slab-on-grade foundations performed well at low liquefaction levels, and better than concrete perimeter foundations overall. This is because the slab foundation is stiffer than the concrete

perimeter foundation system, and at low levels of liquefaction it can better resist the smaller loads and deformations. The median values for equivalent floor slope and differential settlement were below the DBH criteria for houses needing repair of 1/200 floor slope or 50mm differential settlement.

- At higher liquefaction levels, slab-on-grade foundations are more likely to need repair or rebuilding. Their weak slabs are not stiff enough to withstand the high loads at higher levels of ground damage.
- Ribraft and piled foundations performed well up to moderate levels of liquefaction, performing the best of the four foundations types overall. Only a few foundations required repairs based on the DBH criteria at these liquefaction levels. The stiffer and stronger ribraft slab and the better-supported piled foundations are more competent at resisting the loads and deformations associated with liquefaction.
- Lateral spreading was found to cause severe foundation damage in all foundation types, with maximum differential settlements for houses suffering significant lateral spreading of 474mm, 202mm, 182mm and 250mm for concrete perimeter, slab-on-grade, ribraft and piled foundations respectively. This showed that even the better ribraft and piled foundations were not strong and stiff enough to resist severe structural damage under large liquefaction-related loads.
- The data suggests that the weight of the building does not affect the level of foundation or superstructure damage, which agrees with the trends in the data in Chapter 3.

5 Invasive Inspections

5.1 Introduction

Following the detailed, non-invasive inspection of houses after the 22nd February 2011 earthquake, it was decided to conduct a small number of more detailed, invasive inspections of houses and their foundations. The aim was to explain some of the results and trends seen in the analysis of the data from the detailed inspections in Chapter 4.

The invasive inspections were achieved by inspecting houses in the residential Red Zone (New Zealand Government, 2011) that were to be demolished. A standard detailed inspection, as in Chapter 4, was conducted prior to demolition, with damage to the house and surroundings, and floor levels and slopes recorded. Then during demolition, details of the house and foundation construction were recorded, and evidence for explanations behind the damage seen was investigated.

A total of seven houses were inspected, evenly divided between concrete perimeter and slab-on-grade foundations. All seven houses were located close to each other, in a suburb that suffered severe liquefaction and lateral spreading in multiple seismic events during the Canterbury earthquakes.

5.2 Methodology

5.2.1 *House Locations*

Seven houses were inspected in the suburb of Dallington in Christchurch, with their locations pictured in Figure 5.1. The houses were selected evenly between the two main types of foundations used in residential houses in New Zealand. Three had concrete perimeter foundations, three had slab-on-grade foundations and one had a combination of the two. All houses suffered significant land damage from liquefaction and lateral spreading effects from multiple earthquakes during the Canterbury earthquake sequence, due to the high seismic demands from the 22nd February 2011 Christchurch earthquake, and the poor soil conditions, as discussed in Chapter 4, Section 4.4.

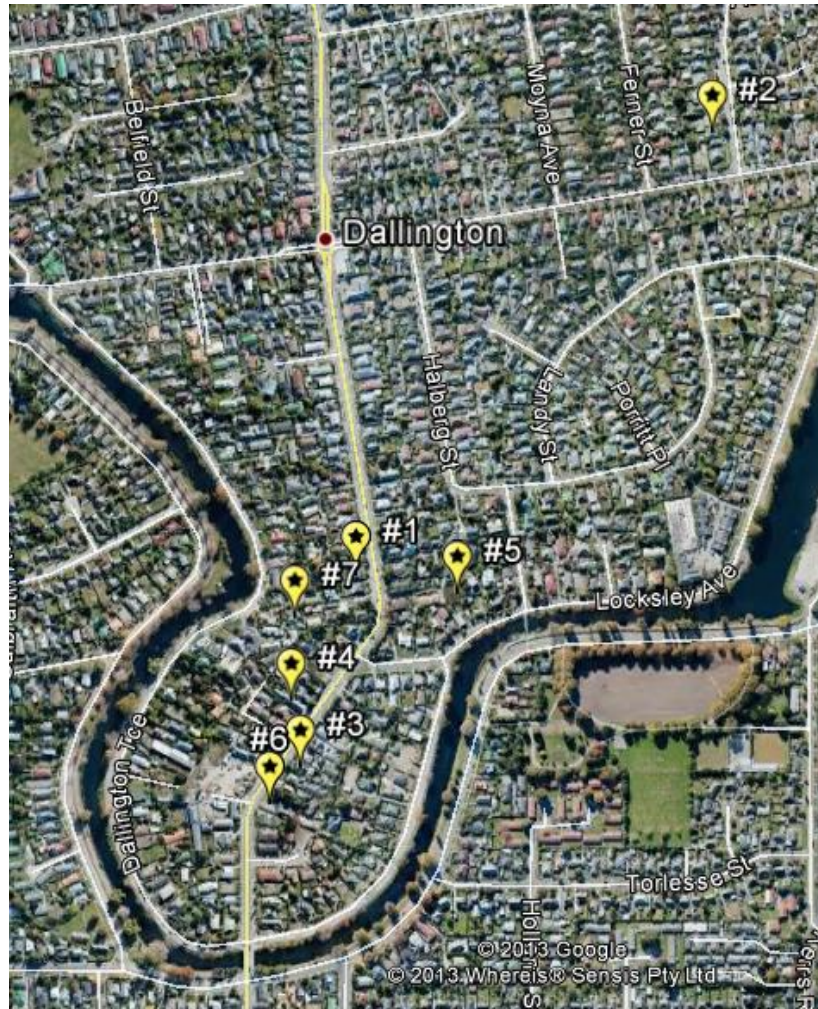


Figure 5.1. Location of inspected houses.

5.2.2 Inspection Methodology

The invasive inspection of houses was done in two parts. This included a pre-demolition inspection followed by observation during the demolition.

5.2.2.1 Pre-Demolition Inspections

Prior to demolition, a detailed internal and external inspection was carried out, the same as for the detailed inspections following the 22nd February 2011 earthquake. This process is outlined in Chapter 4, Section 4.3, and involved recording details of the damage to the house, including the exterior cladding, interior linings, foundations and structure as a whole, damage to surroundings and the land. Floor levels and tilts were also measured.

A higher level of detail was observed in the data recorded for these few inspections, with more values recorded for floor levels and tilts, and more detailed descriptions of the damage and deformation characteristics of the houses and foundations. As the houses being inspected were known prior to their inspection, approximate floor plans were drawn from their corresponding Google Earth images, with

room layouts added during the inspections, to allow greater accuracy and detail in recording damage and floor measurements.

5.2.2.2 Demolition Observations

During demolition, the detailed pre-demolition data was added to. Construction details of the actual foundations were recorded, along with ground conditions under the foundations. These factors were related back to the damage and deformations recorded during the pre-demolition inspection in an attempt to connect the two and suggest possible causes and mechanisms that produced the deformations and damage observed.

5.2.3 Data Processing

Upon completion of each inspection, a full report was completed for each house. These outlined the overall details of the house, construction details of the foundation and the damage and deformation for the surrounding property, the superstructure, foundation and the overall structure. Pictures were included to illustrate the damage. These reports can be found in Appendix T.

5.2.3.1 Contour Plots

Contour plots were produced for each house in ArcGIS to allow easy visual comparison of the deformation in each foundation. These were done by first inputting the Zip Level floor level measurements into Google Earth, in their correct spatial positions over the floor plan of the house. This file was then converted to GIS and the New Zealand map coordinate system, using the NZGD 2000 Mt. Pleasant Circuit, as this was the closest and most accurate. The floor elevation values of the data points were then used to interpolate between with a topographic to raster conversion in ArcGIS, creating the contour plots of the floor levels shown in this Chapter.

Contours were created in 10mm increments. The actual datum for the contour lines is arbitrary, due to the nature of the zip level floor measurements. During the inspections, the zero point in each house was chosen to be a convenient corner for starting at, and was not always the highest or the lowest or the exact middle height of the floor elevations. Because of this the contour values are only provided as a reference, to give the height difference between and two positions on the floor plan.

For all seven houses, the contour at the mid-height of the floor (whatever value it might be) was given the same exact shade of green, with the same exact colours also used for each 10mm increment either up or down in floor level. Contours moving towards blue in colour were used to denote increasing floor height, while contours moving towards red denote decreasing floor height. This allows easy visual comparison between the seven houses as to which foundations deformed the most and in what way.

5.3 Foundation Construction Details

5.3.1 Concrete Perimeter

There was a large variety in the strength and construction characteristics between the three concrete perimeter foundations that were inspected, but a number of common features were also found.

5.3.1.1 Perimeter

Two of the three concrete perimeter foundations inspected had no reinforcing at all and had quite poor construction, despite being constructed in the 1930-59 period, after the introduction of housing standards. They both used uniformly graded round aggregate, and had very boney concrete mixes. As shown in Figure 5.2, there were extensive areas on the inside of the perimeter walls showing large gaps around the aggregate where very little cement was present.

The same two unreinforced concrete perimeter foundations also used large round stones of 200-300mm diameter as fill right through the height of the wall, as shown in Figure 5.3. When the perimeters were lifted up with the excavator by the contractor, the concrete broke smoothly around these stones, indicating they provided a plane of weakness within the concrete wall (Figure 5.3). Similar construction techniques were found in a number of the concrete perimeter foundation houses inspected for the detailed inspections in Chapter 4, where a variety of fills were used, such as bricks and mortar and other pieces of concrete rubble. It is suggested though that the round stones in these two foundations would be the worst case for fill, as they provide no chance for mechanical interlock of the concrete cast around them, and act as planes of weakness.



Figure 5.2. Boney concrete mixes in concrete perimeter walls, (left) large area with little cement fill and (right) close up showing uniform round aggregate.



Figure 5.3. Large 200-300mm diameter stones used as fill in concrete perimeter, (left) showing unreinforced perimeter wall and (right) clean break-up around stones.

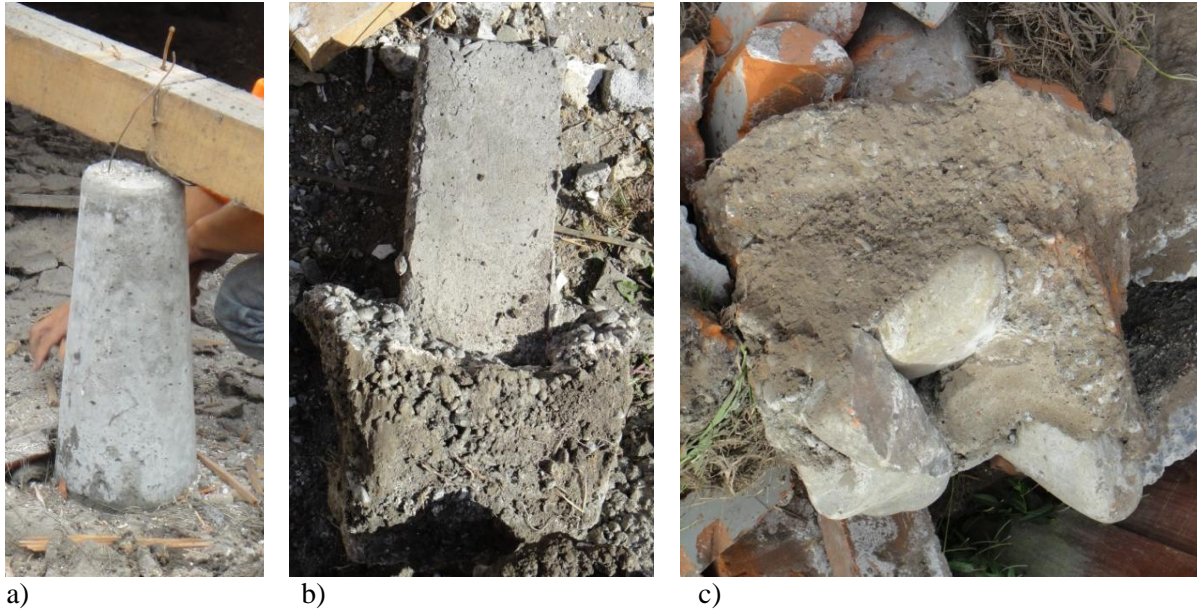
The dimensions of the concrete perimeters were similar in all three foundations inspected. They ranged from 130-250mm in width, and 600-700mm total depth. Generally 200-300mm of the perimeter was cast into the ground, with 300-400mm out, consistent with the standards at the time.

The fourth perimeter, in the house with the mixed foundation types, was constructed of unreinforced masonry, extending 200mm into the ground, and actually continued the full height of the wall as the exterior cladding. It was not connected at all to the interior floor structure.

5.3.1.2 Piers

The piers used in each of the houses inspected were all similar. They were either round or square in cross-section and mostly precast, although one house had square cast-in-place piers (Figure 5.4, (a) and (b)). Most piers were cast into the ground by 100-250mm, with footings that were 200-400mm deep and 300-400mm in width. One foundation used large stones as fill in these footings, like in the perimeter walls (Figure 5.4.(c)).

The majority of piers were connected to the subfloor framing with wire hooks that were cast into the piers as in Figure 5.4.(a), although in one house the hooks were not connected at all, with the floor just sitting on top of the piers.

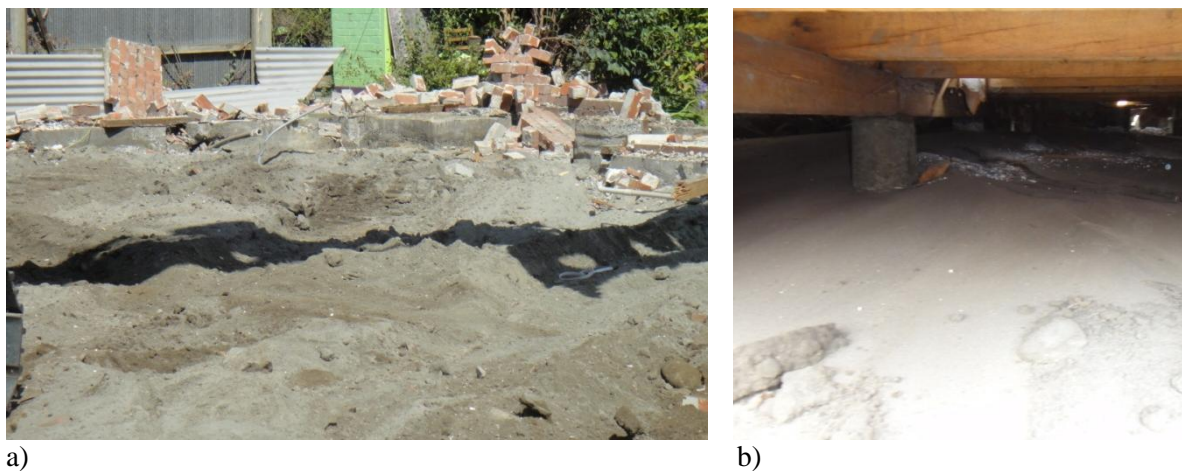


a)
 b)
 c)
Figure 5.4. Pier details, a) round pier connected to floor bearer with wire hooks, b) square cast-in-situ concrete pier with typical footing, c) large stones used in footing.

5.3.1.3 Ground Conditions

Shallow test pits were dug under the three concrete perimeter foundations to determine the underlying ground conditions and offer some clue as to how the ground may have behaved. Each of the sites had somewhat different conditions, which is to be expected in Christchurch, although there were characteristics that were the same at all three.

At House #1 there was extensive sand ejecta under half the foundation, up to 200-300mm thick, as shown in Figure 5.5, indicating the likelihood of poor ground conditions below. The soil below the original ground surface was silty sand, very similar to the ejecta at the surface, with no overlying topsoil. This extended to the bottom of the test pit at 1.8m. This is similar to the CPTs analysed from Dallington in Chapter 4, Section 4.4, and indicates that very severe liquefaction was possible, due to the loose, susceptible soils present right to the surface (Figure 5.7.(a)).



a)
 b)
Figure 5.5. Extensive sand ejecta under House #1, a) sand ejecta spread under the whole foundation and b) sand boils up against concrete piers.

House #2 had far more competent soils close to the ground surface, and no surface liquefaction ejecta was found under the house. From ground level, there was topsoil to a depth of 0.5m, which was underlain by a cohesive but low-plasticity clayey silt to 1.7m (Figure 5.6). Below this was the same grey sandy-silty material seen at the first site to the bottom of the test pit at 2.2m. This sand was moist and when vibrated in the hand quickly became wet on top, indicating susceptibility to liquefaction (Figure 5.7.c)).

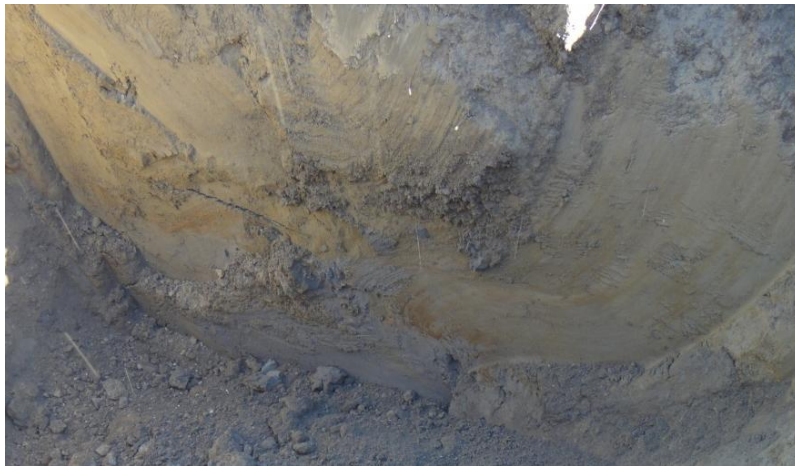
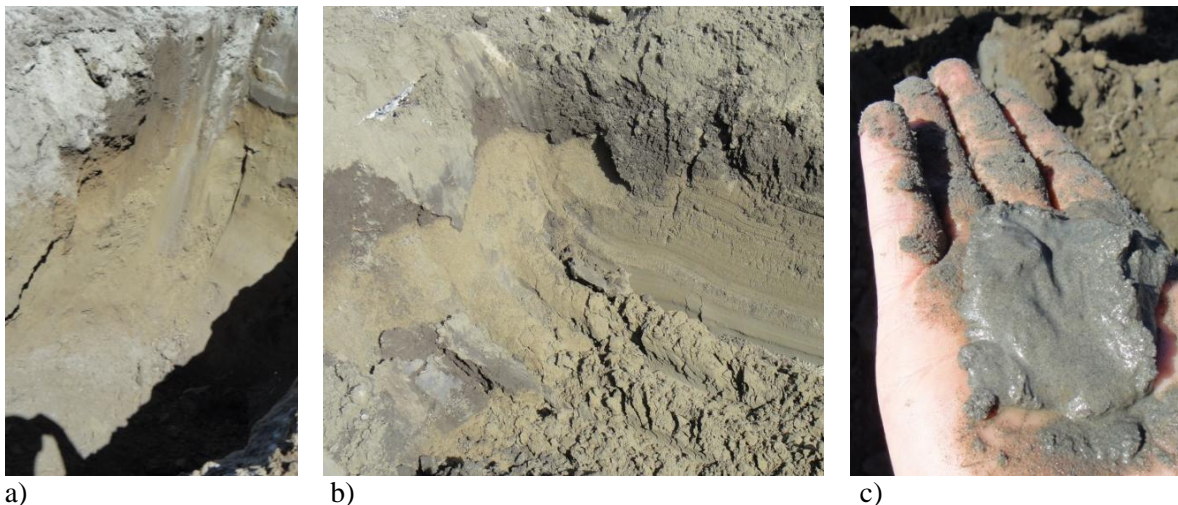


Figure 5.6. Competent crust layer of cohesive, low-plasticity clayey silt at House #2.

The third house also had extensive liquefaction ejecta under half of the house. Excavation showed a thin 200-300mm layer of topsoil below that, which was underlain by the same silty sand, down to ~0.7m at the bottom of the shallow test pit (Figure 5.7.b)).

The presence of the loose silty sand at all three sites at shallow depths shows how poor the general ground conditions were in the suburb where the inspections were conducted. However, the presence of the 1.7m thick crust layer at the second house (Figure 5.6) prevented the liquefied material from breaking the surface there.



a)

b)

c)

Figure 5.7. Soil excavations, a) 1.8m of silty sand under house one, with light grey surface ejecta at top b) shallow topsoil (dark brown) underlain by silty sand with surface ejecta on top, c) wet silty sand after shaking.

5.3.2 Slab-on-grade

As with the concrete perimeter foundations, there were similarities and differences in the four slab-on-grade foundations that were inspected.

5.3.2.1 Foundation Wall

In all four slab-on-grade foundations inspected, the perimeter foundation wall was reinforced with at least two deformed bars. The size and arrangement of these varied from D10 to D16 bars, placed anywhere in the wall, though generally there was at least one near the top and one near the bottom. Three of the foundations also had tie bars between the foundation walls and slab itself (Figure 5.8.(b)).

The walls ranged from 200-300mm wide, and 500-700mm deep, and could be either plain rectangular in cross section or with a wider un-boxed base, as shown in Figure 5.8.(a). One of the foundations also had intermediate foundation walls which were narrower and shallower than the perimeter ones, under each of the major load-bearing walls.

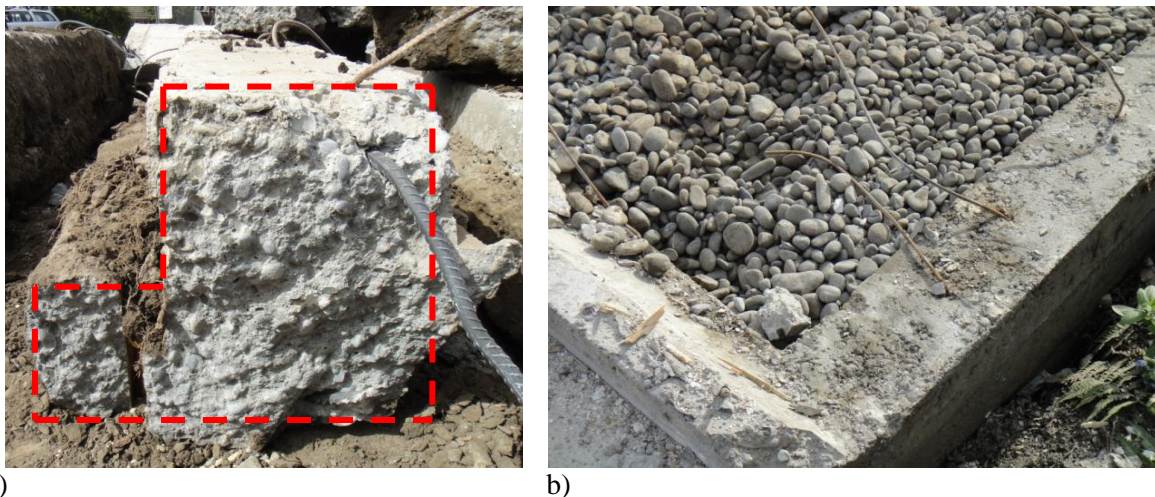
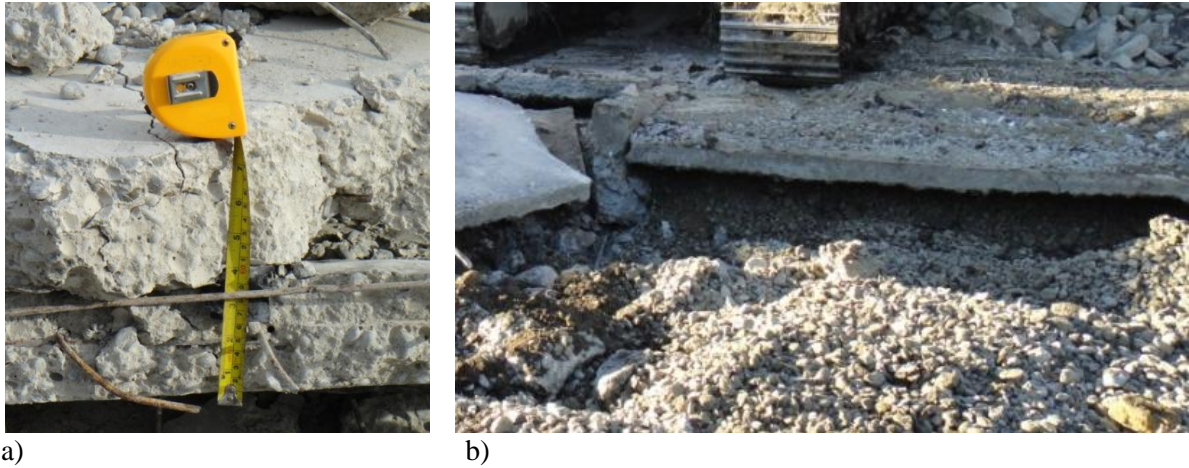


Figure 5.8. Slab foundation details, a) perimeter foundation wall cross-section, showing reinforcing bars protruding, b) foundation wall from above showing tie bars into slab and round river-run fill.

5.3.2.2 Slab

Three of the slabs inspected were very similar, ranging from 95-160mm in thickness, and reinforced with steel wire mesh (Figure 5.9.(a)). The other slab foundation was unreinforced and close to 100mm thick across the whole floor (Figure 5.9.(b)), and overlain by a 50mm thick wood-chip/cement mixture. This somewhat lesser construction was due to the house being constructed in the 1930-59 period, prior to the introduction of residential slab foundation construction standards. This was made up for however by intermediate foundation walls and a more competent fill.

The concrete quality varied from slab to slab, from weak concrete with round aggregate that broke up easily to stronger mixes with good well-graded angular aggregate.



a) b)
Figure 5.9. Slab details, a) two layers of concrete floor slab showing thickness and wire mesh protruding, b) 100mm thick slab over fill.

5.3.2.3 Fill

The three newer foundations had the same fill under the floor slab. This was river-run, round stone tailings, 20-50mm in diameter (Figure 5.8.(b)). The nature of this gap-graded material is that it is reasonably loose, and cannot be compacted far beyond its loosely deposited density. The fourth, older foundation had AP40 or AP60 or a similar mix fill, uniformly graded angular gravel. This material has much better interlock than the round stone tailings used in the newer three foundations.

5.4 Summary of Performance

Table 5.1 shows the four measured foundation damage indices used for the analysis in Chapter 4, for each of the foundations inspected.

Table 5.1. Summary of floor levels and deformation indices for the inspected houses.

House	Foundation type	Differential Settlement	Equivalent Slope	Maximum Slope	Maximum Local Slope
1	Concrete Perimeter	150mm	0.012	0.014	0.023
2	Concrete Perimeter	70mm	0.005	0.016	0.023
3	Concrete Perimeter	200mm	0.018	0.031	0.038
4a	Brick Perimeter	174mm	0.011	0.015	0.026
4b	Slab-on-Grade	232mm	0.011	0.018	0.024
5	Slab-on-Grade	104mm	0.009	0.015	0.030
6	Slab-on-Grade	180mm	0.011	0.016	0.017
7	Slab-on-Grade	20mm	0.001	0.003	0.007

5.4.1 Concrete Perimeter Houses

5.4.1.1 House #1

House #1 was very severely damaged. The building had an overall differential settlement of 150mm to the Southwest and was tilting in a relatively uniform nature, as shown in Figure 5.10, though there were a couple of areas of local floor humping in the Southwest area of the house. This is shown in Figure 5.10 as the gradient of the contours flattens out at the Southwest corner.

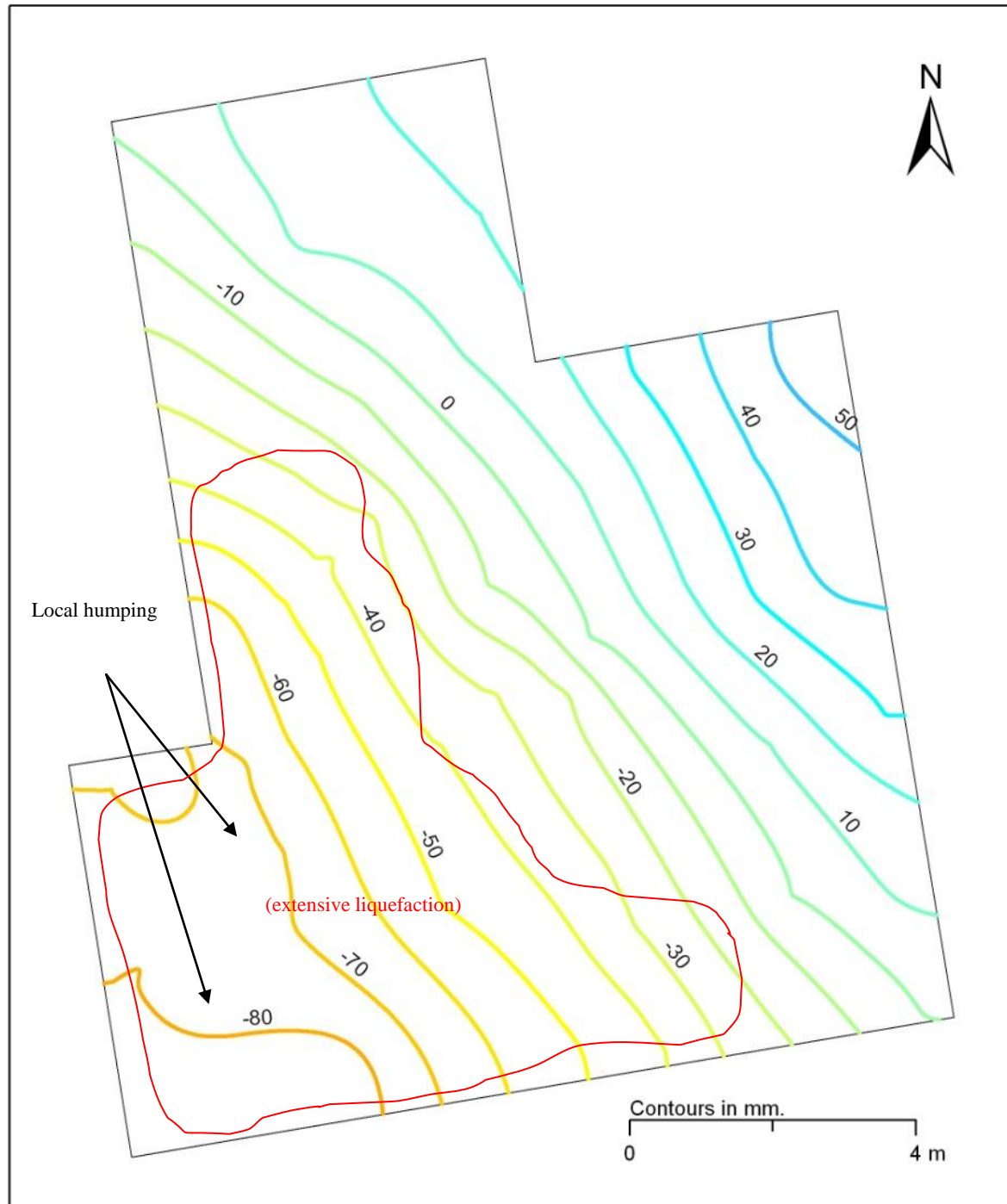


Figure 5.10. House #1 floor plan outline showing floor level contours in 10mm increments, with the area of extensive liquefaction outlined in red.

Extensive sand ejecta was found under the Southwest half of the house, up to 200-300mm thick. This was under the locations of local floor humping, and indicates that the ejection of sand and water could have pushed up the shallow piers causing the humping. The location of the majority of sand ejecta was also located where the house was settling towards, indicating that the bearing loss was greater there. There was no other apparent reason for the house to settle one way or the other.

Despite the uniform tilting of the foundation and floor, the superstructure suffered significant damage. There was evidence of racking in a number of locations, with many doors and windows that would not close properly. At the Northwest corner of the house, a large section of the top of the exterior brick cladding had somewhat broken away from the rest of the house which was tilting away from it. This caused horizontal cracks to open up in the brick work from the base of the wall to 10mm width at the top of the wall (Figure 5.11.(b)). This suggests that the superstructure was not strong enough to fully rotate and tilt uniformly with the foundation. As a result the stiff elements (the brick cladding) suffered damage. Due to the flattening out of the floor in the Southwest corner, the walls at this corner would have pushed up against the rest of the structure trying to tilt over with the foundation. This resistance to movement would have been transferred via load paths through the roof structure causing the racking of internal walls and the pushing out of the Northwest corner as described.

The concrete perimeter of this house was very poorly constructed. It was unreinforced, with round aggregate and a dry concrete mix, and had large round stones as fill. Despite this poor construction, it only suffered minor damage, with a total of five cracks around the entire perimeter, all of which were 1-2mm wide. This low level of damage is attributed to the uniform tilting of the structure, though why it tilted uniformly is harder to explain. Given that the superstructure resisted the uniform tilting and suffered significant damage it would make sense that the perimeter did not tilt uniformly either. The very soft soil may have allowed the uniform tilt of the foundation, and have kept the loads on the foundation low enough to prevent significant cracking and separation.

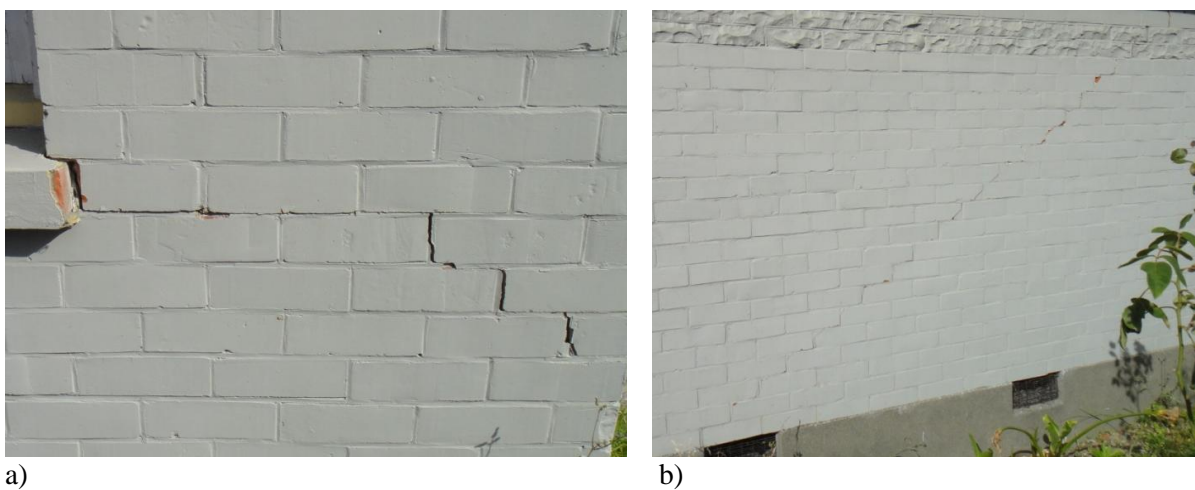


Figure 5.11. Structural damage, a) diagonal crack in bricks on North wall at Northwest corner, b) foundation to roof diagonal crack on West wall of Northwest corner.

5.4.1.2 House #2

This house had a unique mode of deformation caused by the settlement of two small extensions on the Southwest and Southeast arms of the 'T' shaped house (Figure 5.12).

The house was generally level along its major axis, North to South (Figure 5.13) with only 10-20mm difference in floor level between the middle of the North and South walls. The perimeter foundations of the extensions, shown in Figure 5.13 with dashed lines, were not structurally attached to the rest of the house, but sitting beside it, as shown in Figure 5.12 and Figure 5.14.(a). This meant that the structure could not act as a cohesive system, and these extensions were not aided by the flexural strength and stiffness of the main body of the foundation. This meant that the foundations of the extensions settled more than the main body of the house. They dragged the rest of the Southern part of the house down with them, as the sections of foundation were still connected through the superstructure, which still acted as one unit. This caused significant hogging of up to 50mm across the Southern part of the foundation, as shown by Figure 5.12 and Figure 5.13, and resulted in significant structural damage in this area. Two doorways showed evidence of racking, and major vertical structural cracks opened up in the walls either side of where the East extension joined to the old house, one of which is shown in Figure 5.14.(b).

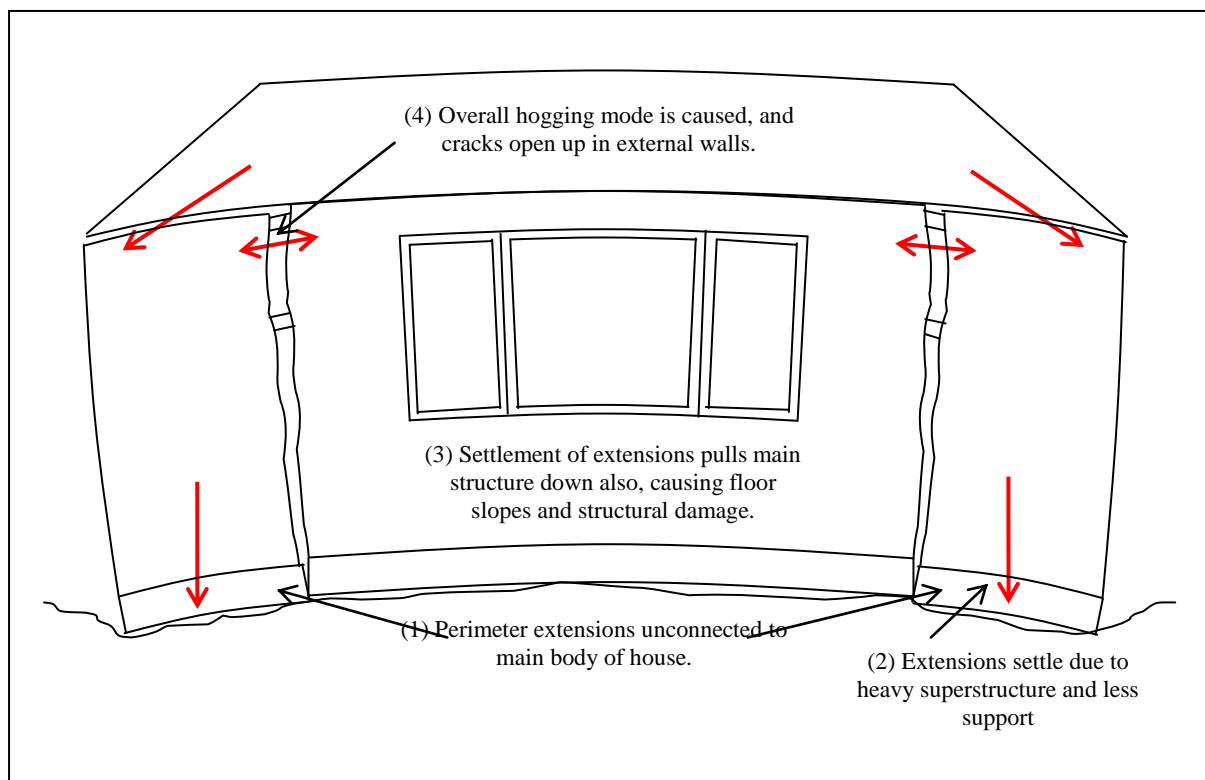


Figure 5.12. Schematic elevation A-A showing the deformation mode of House #2, looking towards the North from the back of the house.

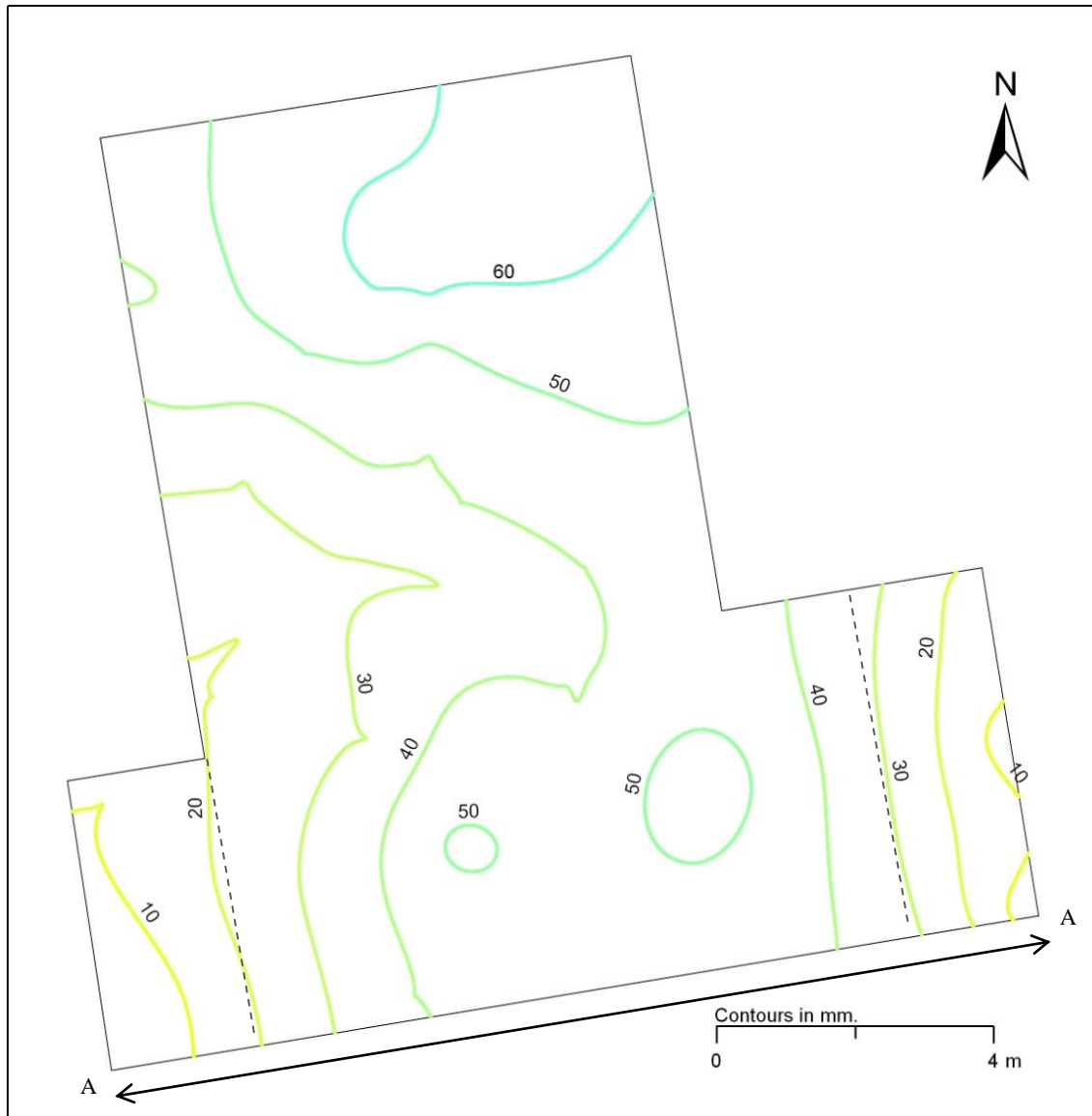


Figure 5.13. House #2 floor plan outline showing floor level contours in 10mm increments and extensions with dashed lines.



a)



b)

Figure 5.14. a) Perimeter foundation showing old foundation in blue and new extension in red, with no connection between the two, b) structural crack on North side of East extension, opening up to 30mm at roof level.

The concrete perimeter was well constructed, with two R16 reinforcing bars. Because of this there was very little damage to the perimeter itself, despite the significant hogging, and visible damage was concentrated in the superstructure along the Southern section. There was little sand ejecta under the house, due to the 1.7m thick crust layer found. In the absence of the extensions, overall damage and deformation of the house may have been only low to moderate, as indicated by the low level of damage to the main body of the original house. However, the unattached perimeters in the extensions, combined with heavy concrete block walls, and a heavy tile roof, caused significant structural damage to this house.

5.4.1.3 House #3

House #3 performed very poorly. Aside from the high levels of liquefaction, this is assumed to be due to the poorly constructed foundations. The concrete perimeter was un-reinforced and constructed with poor quality concrete. Like in House #1, the aggregate was uniformly graded round stone, with a very dry mix, as shown in Figure 5.2. Large stones of 200-300mm diameter were also used as fill in the perimeter, as in Figure 5.3. The internal piers supporting the floor were also constructed poorly. Under one half of the house they were not attached to the sub-floor structure at all, leaving the bearers just sitting on top. In some cases, the piers were not cast into the ground at all, but were sitting on top of their footings (Figure 5.15.(b)). The ground conditions were also very poor, with silty sand (i.e. liquefiable material) found below a thin 200-300mm layer of topsoil (Figure 5.7.(b)).

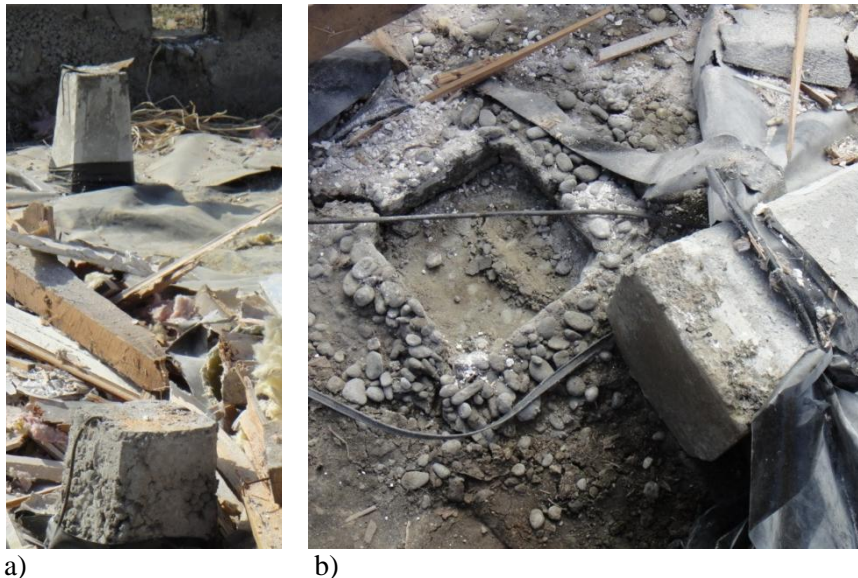


Figure 5.15. Pier details, a) upright piers showing wire hooks bent down and not used, b) pier cast only level to the ground, easily knocked out.

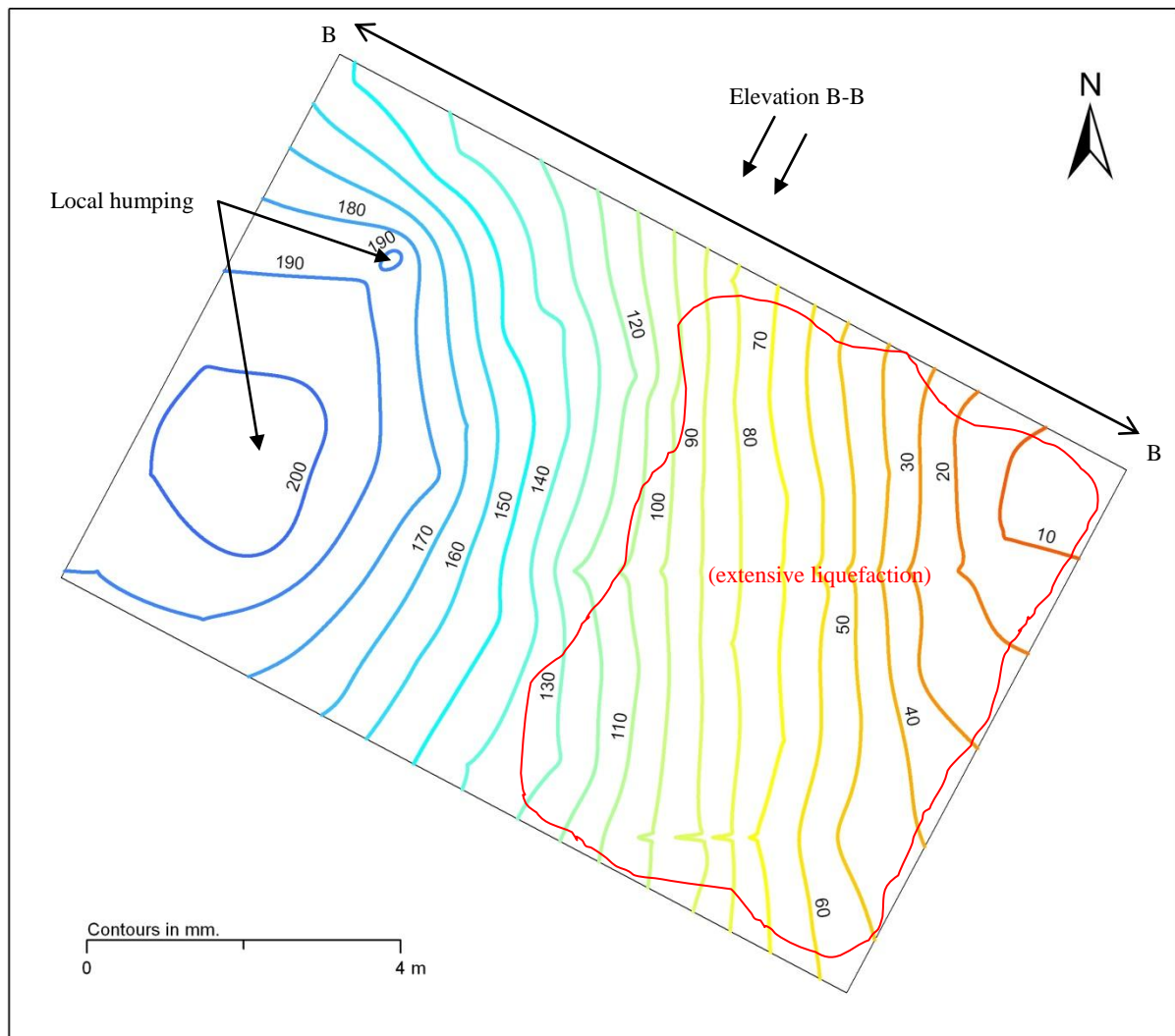


Figure 5.16. House 3 floor plan outline showing floor level contours in 10mm increments.

The building had an overall differential settlement of 200mm from the West corner down to the East corner, as shown in Figure 5.16 and Figure 5.17. This settlement was quite uniform, apart from room humping in two bedrooms at the West end, shown in Figure 5.16 and Figure 5.17. This local room humping caused some extreme slopes of 2.2° , or 0.038m/m . The general floor slopes in the direction of overall differential settlement were between $1\text{--}1.5^\circ$.

The most severe damage was to the foundation. The superstructure remained relatively intact and moved somewhat independently of the foundations, while the unreinforced concrete perimeter completely broke up in multiple locations and settled differentially (Figure 5.17). In addition to the detached piers, along two thirds of the Northeast wall (about 6m) the perimeter broke up during settlement and was completely separated from the superstructure above, leaving a gap of up to 70mm between the two. This is shown in Figure 5.17 and Figure 5.18. This has caused the wall above to bend flexurally, and has resulted in eight small tension cracks from hairline to 3mm in width along the base of the wall.

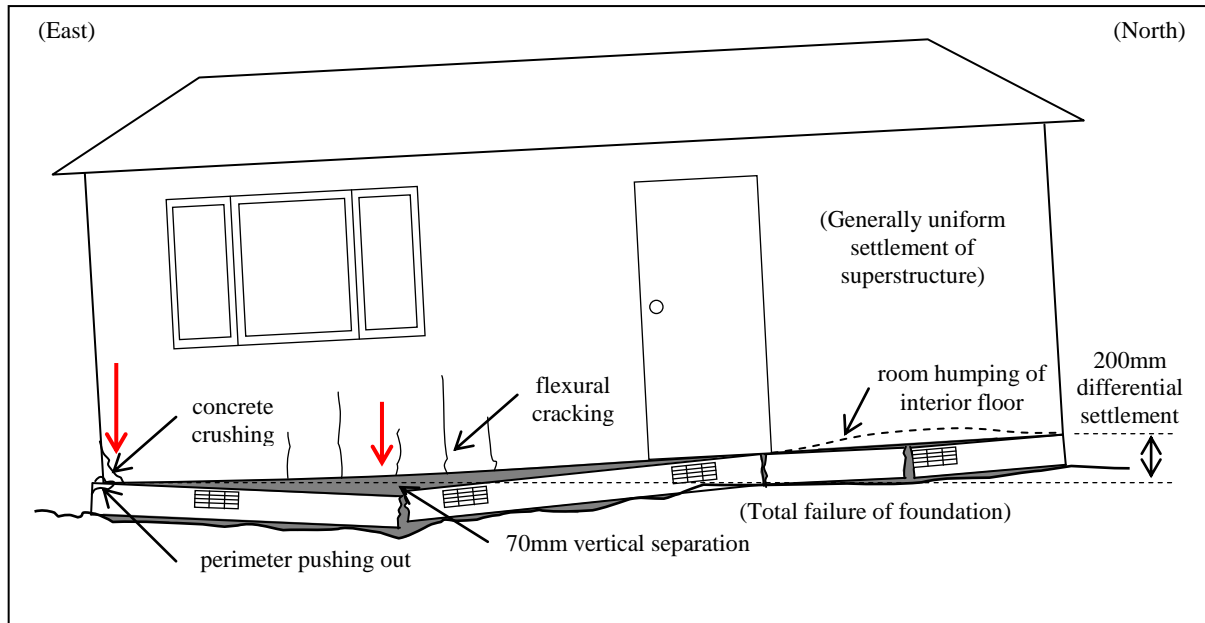


Figure 5.17. Schematic elevation B-B of the Northeast side of House #3, showing deformation and damage, including the settlement and failure of foundations, uniform tilt of superstructure and separation between foundation and superstructure causing flexural cracking in external cladding.



Figure 5.18. East corner looking along the Northeast wall showing separation between superstructure and concrete perimeter, and crushing at East corner.

At the East corner, the perimeter has pushed out from under the house by 40-60mm (Figure 5.17 and Figure 5.18). At this corner, and also the Southeast corner, the superstructure is pushing down onto the perimeter, causing spalling at the top of the perimeter at the corners. This is matched at the opposite West corner, where the superstructure has lifted off the perimeter by 30mm (Figure 5.19). This is a result of the perimeter settling differentially but non-uniformly to the East while the superstructure tilted uniformly in the same direction.



Figure 5.19. West corner showing uplift of superstructure and separation between foundation and superstructure caused by tilting.

The superstructure, along with the timber floor and sub-floor, separated from the foundation and tilted almost as a rigid body, giving the overall differential settlement of 200mm (Figure 5.17). This near rigid body movement meant that damage to the superstructure was only low to moderate throughout the house. Outside, apart from the eight small flexural cracks, a number of minor cracks were seen in the stucco cladding at window corners, and there were a small number of vertical cracks and separation between door frames and walls. Inside the damage was minor, with hairline cracks in the plaster in most rooms. A few doors stuck open due to local room humping, but this was confined to a couple of rooms, and there was no apparent racking of door frames.

In contrast to the good performance of the superstructure, the broken and uneven differential settlement of the perimeter beneath it caused gaps to open up between the foundation and superstructure, and the crushing of concrete in some locations. This deformation was the combination of a weak concrete perimeter, lack of connection between foundation and superstructure and the severe liquefaction during multiple earthquakes. The superstructure performed very well given the loading conditions it was subjected to.

5.4.2 Slab-on-Grade Houses

5.4.2.1 House #4

House #4 had complex behaviour and deformations, due to its large floor plan, and combination of different foundation types. The Southeast half of the house (seen in Figure 5.20) will be examined here as a standard slab-on-grade house, attached to another house. This half of the house, referred to from here on as the south slab, suffered severe structural damage and deformations, caused by the settlement of the foundation, and the connection to the other half of the house.

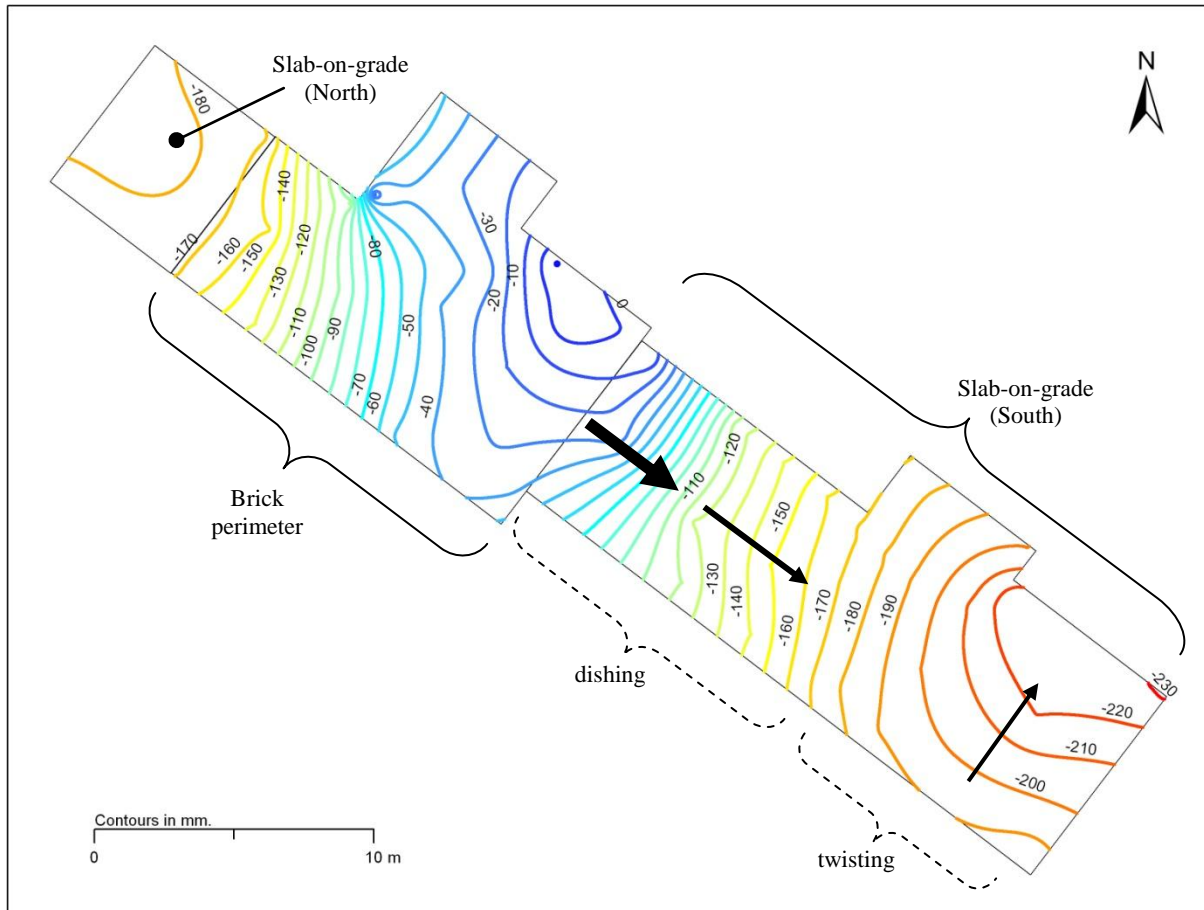


Figure 5.20. House 4 floor plan outline showing floor level contours in 10mm increments.



a)



b)

Figure 5.21. Structural damage, a) racking of door frame, b) cracking and mis-alignment of wall from concrete slab foundation.

From the connection between the two halves of the house, the south slab had an overall differential settlement of 232mm to its Southeast end. This differential settlement was not uniform. Along the length the south slab, there is significant dishing over the northern half, shown by the change in spacing between the contour lines in Figure 5.20. The Southeast quarter of the south slab has twisted and is tilting to the Northeast, perpendicular to the rest of this slab section, shown by the orange and red contour lines in Figure 5.20.

The dishing of the south slab foundation has pulled the superstructure down with it, causing significant racking and the mis-alignment of a number of door and window frames, as seen in Figure 5.21.(a). The dishing also caused cracking and mis-alignment between the Southwest wall and the

foundation along the whole Southwest wall of the south slab, shown in Figure 5.21.(b), and nail popping and buckling of the interior plasterboard on the Southwest wall in the garage, at the South end of the wall (Figure 5.22.(a)). The twisting at the South end was not manifested in superstructure damage, as this was the garage, with no internal walls.



Figure 5.22. Structural damage, a) buckling of plasterboard on Southeast wall in garage and separation between two halves of house on b) South side and c) North side.



Figure 5.23. The un-cracked south slab being lifted (causing cracking), showing hard fill layer underneath.

Despite the significant differential settlement, dishing and twisting, there were no visible cracks found in the top of the concrete slab foundation (Figure 5.23). Even after the superstructure had been totally pulled off the foundation, and the excavator had driven all over it, the slab showed no signs of damage. It wasn't until the foundation was pulled up that it broke apart. This may have been due to the fact that the deformation occurred along the house, and the flexural demands (in terms of curvature) on the foundation were not great due to its very slender profile in this direction (although the underside of the slab which experienced the tension stress from the dishing could not be inspected

for cracks), but it also suggests that competent slab-on-grade foundations can resist significant deformations without suffering serious damage in terms of cracking, yielding and rupture. The slab was 120-150mm thick with steel wire mesh reinforcing, and was tied into competent, reinforced foundation walls. However, an un-cracked slab is not much use if the superstructure above is severely damaged.

The most severe structural damage occurred at the interface between the slab and perimeter sections of the building in the middle of the house. The two parts of the house were ripped apart, due to their incompatible deformation modes. While the Southeast half of the house was tilting severely to the Southeast, as described above, the Northwest half of the house was tilting the opposite way. This caused 50-100mm of separation between the two halves of the house, as shown in Figure 5.22.(b) and (c).

5.4.2.2 House #5

This two-storey house had a high overall differential settlement of 104mm across its widest point. This was largely uniform tilting about its major axis, as shown by Figure 5.25, although there was a one-storey section on the North side about 1.5m wide running across the house that tilted slightly more, causing some hogging in the foundation, which resulted in one hairline crack along this interface (Figure 5.24.(b)). This is illustrated in the yellow contours in Figure 5.25 being closer together, indicating steeper slopes.

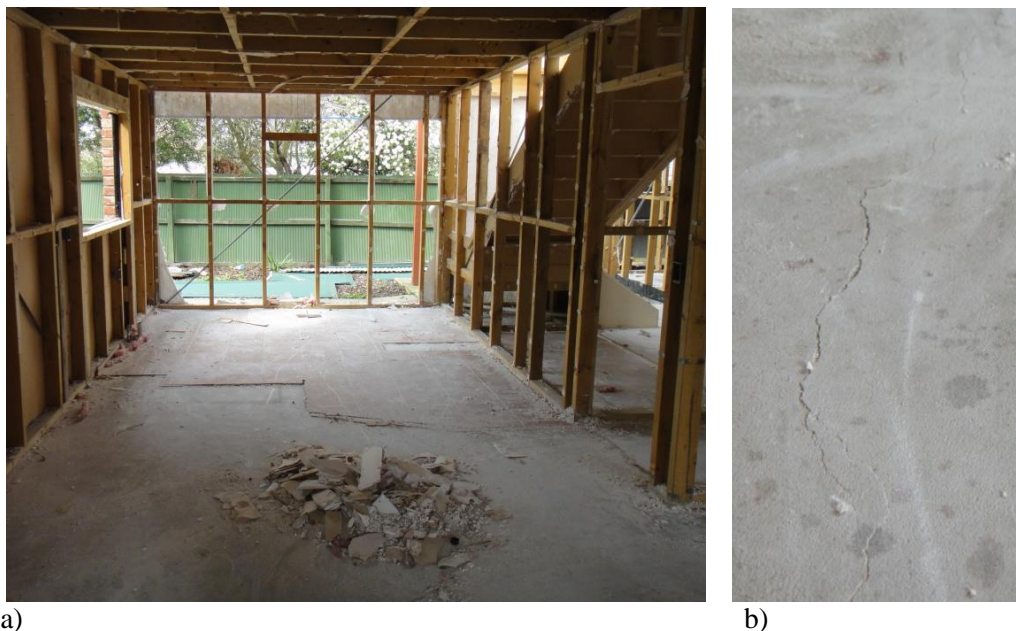


Figure 5.24. a) Overview of slab foundation in living area of House #5, b) The only crack in the floor slab.

Due to the uniform, rigid-block tilting of the house, damage to the superstructure was very low, with only a few instances of cracking in brickwork at window corners, and typical minor damage to plasterboard inside.

This deformation mode is attributed to the well-built foundation of this house, which was on average thicker than the required 100mm; up to 160mm thick in places, and well reinforced with either 665 or 663 equivalent wire mesh. It was also surrounded by wide and deep perimeter foundation walls that were well reinforced (for details see Appendix T). The house had a relatively small footprint, due to it being mostly two-storey, which also would have contributed to the rigid-block differential settlement of the house, as the well-constructed foundation would have been more able to hold together a smaller footprint building.

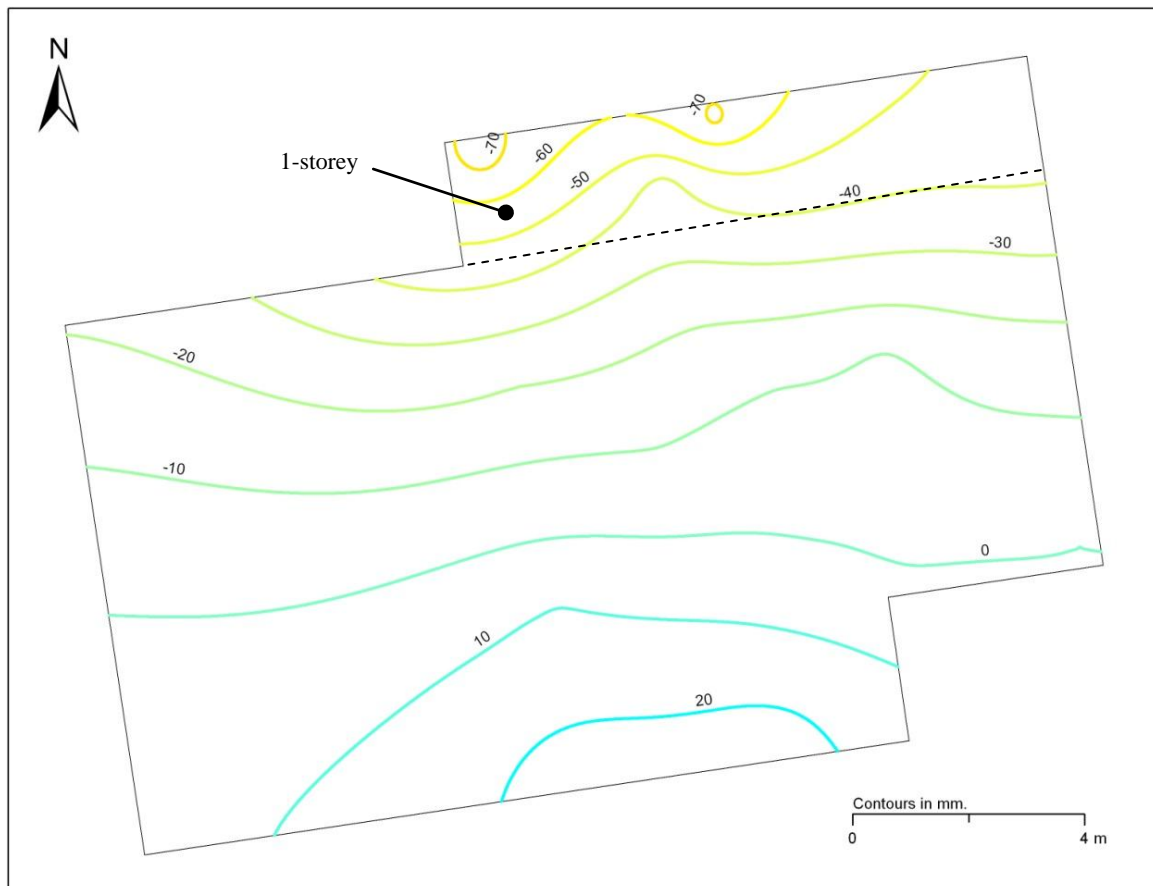


Figure 5.25. House #5 floor plan outline showing floor level contours in 10mm increments.

5.4.2.3 House #6

House #6 also had high overall differential settlement of 180mm from the Southwest corner to the Northeast corner (Figure 5.26). As for House #5, this differential settlement was close to uniform, as shown by the evenly spaced contour lines in Figure 5.26. There was some minor local hogging identified with the smart level in the North section of the house in the North-South direction (for tilt measurements see Appendix T), and there was extensive minor cracking right across the floor slab throughout this area of the house, shown in . However, these cracks were at most 1mm wide, and were not serious, given the overall uniform nature of the settlement. As the cracks were in all directions, and intersecting randomly, it is hard to say whether they were caused by the permanent differential settlement, or during the earthquake.

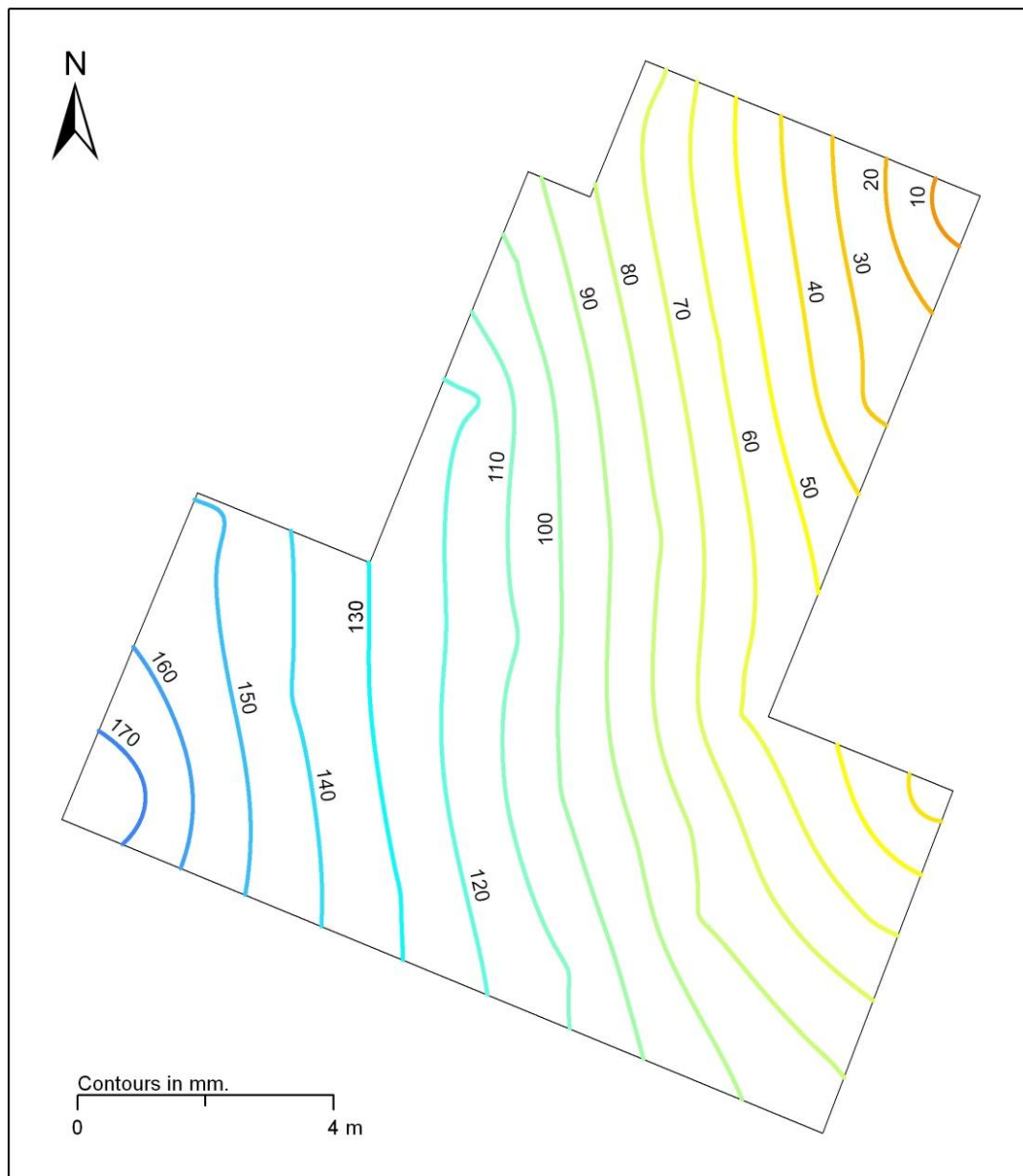


Figure 5.26. House #6 floor plan outline showing floor level contours in 10mm increments.



Figure 5.27. Extensive minor cracking in concrete floor slab in House #6.

Despite the high differential settlement, this house also only suffered low levels of superstructure damage. This is because the uniform nature of the differential settlement does not put any stress on the structural components. Damage to the superstructure was confined to minor cracking in plasterboard inside, and low levels of cracking and separation in a few locations in the external brick cladding outside.

5.4.2.4 House #7

The overall structural damage and deformation in this house was low, indicating good performance. Foundation damage was very low, as the building had a total overall differential settlement of only 28mm over the length of the house (Figure 5.29), with a maximum local floor slope of 0.4° recorded in two locations, the rest of the readings being 0° , 0.1° or 0.2° . This was despite being in an area where major land damage occurred.

The main reason for the good performance of this house is that the foundation was constructed very well. Although the floor slab was unreinforced and the minimum of 100mm thick everywhere, it was constructed with good, well-graded, angular aggregate and stayed together well when lifted. The 50mm covering of wood-chip/cement was firmly attached to the slab and would have provided extra stiffness and strength against deformation. The fill under the slab was also very competent AP40 or AP60, uniformly graded angular gravel that was well compacted and provided a solid base for the foundation. The perimeter foundation walls were large and well reinforced, and there were also intermediate foundation walls under every load-bearing wall that provided significant extra strength and stiffness to the foundation system (Figure 5.28).

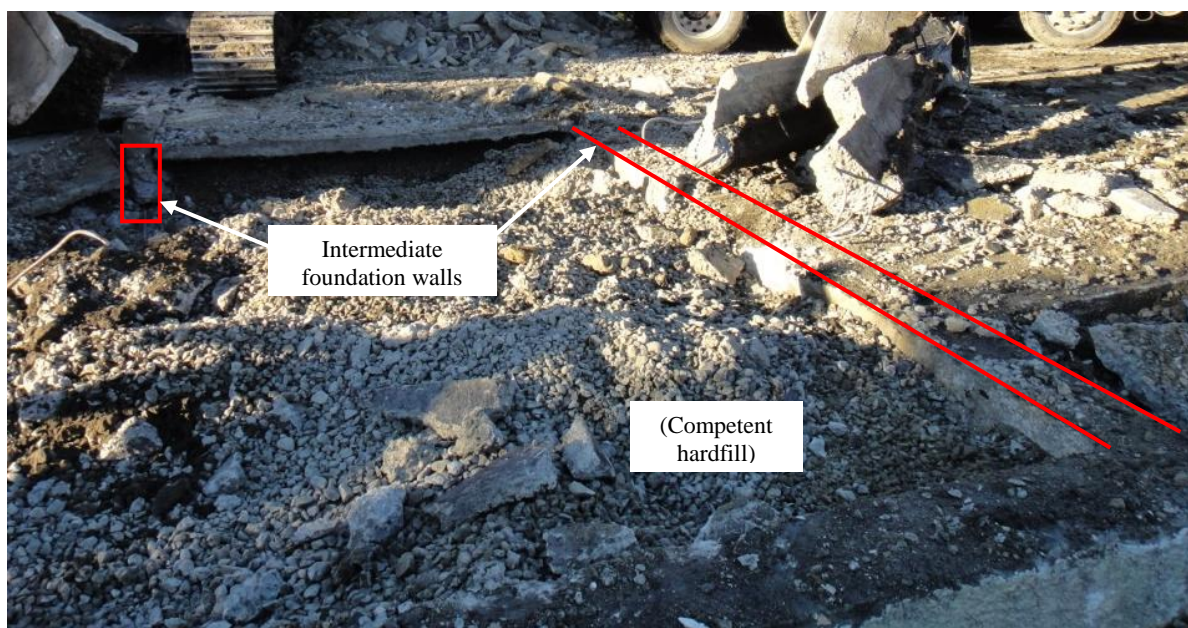


Figure 5.28. Uplifted slab, showing competent hardfill and intermediate foundation walls.

A competent foundation system holds the structure together under the large loads of liquefaction, and if displacements are kept small enough, excessive stresses are prevented from being transferred to the

superstructure. Because of this, the superstructure damage was very low. There was one concrete wall seen behind the fridge space in the kitchen that had minor 0-2mm cracking in it, but apart from that there were only minor cracks inside and out in the cladding.

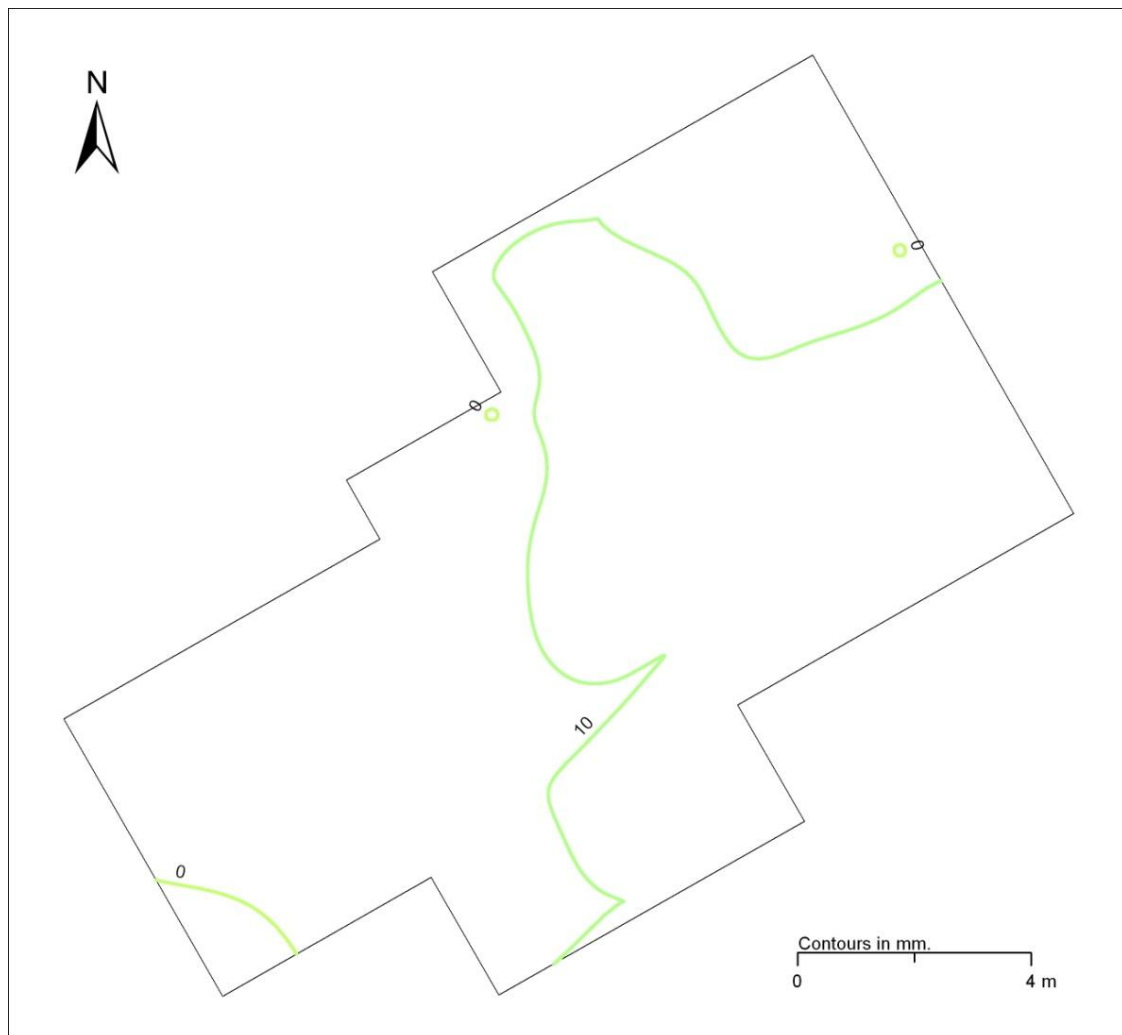


Figure 5.29. House #7 floor plan outline showing floor level contours in 10mm increments.

5.5 Discussion

The seven houses inspected through demolitions provide a good cross-section of the house types that were inspected following the 22nd February 2011 earthquake, and are a good representation of the higher end of damage that was seen in the main detailed inspections in Chapter 3.

A significant variety in foundation construction was seen in the seven houses. Two of the concrete perimeter foundations inspected had no reinforcing, with dry concrete mixes and large round stones as fill, despite being built after the introduction of construction standards. One perimeter foundation was made of brick. There was also one slab-on-grade with no reinforcing in the slab, built prior to the introduction of housing standards. Though it was well constructed with reinforced perimeter foundations and intermediate foundation beams.

There were large differential settlements in all but one of the houses inspected, with the majority of these differential settlements being close to uniform in nature. This appears to be the dominant deformation mode in houses subjected to severe levels of liquefaction.

In the concrete perimeter houses inspected, the damage mechanism was different in each one. House #1 suffered severe superstructure damage due to the high differential settlement of 150mm, but there was only low damage to the unreinforced foundation, due to its uniform tilting. The damage to House #2 was caused by the presence of extensions with foundations which were un-connected to the original footprint. These extensions settled more than the rest of the house causing 50mm of hogging, resulting in significant damage to the superstructure. The foundation of House #3 completely broke up and settled differentially in separate piece due to liquefaction loads on the weak foundation, while the superstructure remained relatively intact and tilting almost uniformly.

The slab-on-grade foundation houses inspected performed better than the concrete perimeter houses, supporting the trends found in Chapters 3 and 4. Two of the four slab-on-grade foundations had significant uniform tilting, with overall floor elevation differences of 100-180mm. However, unlike the two concrete perimeter houses where even uniform tilting resulted in significant structural damage, there was only minor damage to the foundations and superstructure in both slab-on-grade houses. This difference is due to the flexible, non-cohesive foundation system in concrete perimeter houses, where the floors are not rigidly fixed to the piers, and concrete perimeters are sometimes not attached to the interior floor either. This allows local differential settlement causing humping and deformation in interior walls, resulting in damage to the superstructure. In contrast the slab-on-grade foundations remained intact as a cohesive system, tilting uniformly and resulting in little differential deformation and damage.

The slab-on-grade foundations also appeared to resist significant deformation without rupturing, as for House #4 which had severe dishing with no cracking in the slab. Whether this in itself is an advantage is somewhat debatable however, as in the case of House #4, although the slab-on-grade foundation did not crack, there was still severe damage to the structure as a whole. There is little advantage in having an intact slab if the superstructure is destroyed.

The most major structural damage was seen at the interfaces between structural irregularities, such as the un-connected extensions in House #2, and the interface between the slab-on-grade and perimeter sections of House #4. This supports the similar findings of Beattie et. al. (2011) and suggests that these should be avoided where possible.

The connection between foundation and superstructure was found to be very important. The two houses where this was lost (House #3 and House #4) suffered some of the most serious structural damage. In House #3 the concrete perimeter foundation was almost completely separated from the

superstructure. It broke up and settled differentially in separate pieces, while the superstructure remained relatively intact and tilted on top of the foundation, resting on it in some locations and remaining separated by up to 70mm in others. In House #4 the superstructure separated from the slab-on-grade foundation due to differential settlement and dishing, resulting in large damage to the superstructure. In both of these cases, if the two elements had remained attached, they would likely have reduced the resulting deformations and damage seen.

These inspections also highlighted the importance of the ground conditions for concrete perimeter houses and the fill used in slab-on-grade houses. House #2 had the most competent ground conditions of the three concrete perimeter houses and performed the best. The 1.7m thick crust layer under this house prevented the liquefaction from breaking the surface and greatly reduced the deformation and damage to the house. If it wasn't for the extensions creating structural irregularities it is likely the house would have performed very well. Poor ground conditions resulted in large differential settlements and high structural damage in the other two concrete perimeter houses.

6 Modelling

6.1 Introduction

Following the numerous inspections that were carried out for this research, the trends in the behaviour of residential houses, and the liquefaction related deformation features discovered in the previous chapters, it was decided to conduct some simple modelling of the foundations. The purpose of this modelling was to determine the strength of concrete perimeter foundations, and assess their performance under a range of different simplified loading conditions, representative of those that they may have experienced during liquefaction and lateral spreading. These results can then be compared back to the field inspection results to determine if they represent the real-life situation with any accuracy.

The modelling was conducted for concrete perimeter foundation beams, as these provide the clearest examples of the effect of liquefaction and lateral spreading loads on foundations. They also had the greatest variety in terms of the change in standards over the years, and they could be compared against the results from Chapters 3 and 4 and the invasive inspections from Chapter 5.

The modelling was carried out in a number of stages. First the section strengths for a representative cross-section from each age category of foundation were found using the software package Response-2000. Typical house weights and the resulting loads on these foundations were then calculated, along with approximate bearing capacity values for the soils. These properties were then input to a beam model in the software package SAP2000, with joint springs to represent the soil. Soil springs at different locations were reduced in stiffness by varying amounts to represent the different loading conditions. An elastic analysis was used to find the bending moments caused by each of the different bearing loss scenarios. The maximum bending moments were graphed against the calculated section yield capacities to compare the performance of the different foundations.

6.2 Methodology

6.2.1 Section Modelling

The first step in the process was to find the section strengths of the range of different concrete perimeter cross sections. These varied depending on their relevant construction standard, and superstructure construction details. The range of cross sections for concrete perimeters is shown in Appendix A.

Representative cross sections based on these drawings were created in the sectional analysis program Response-2000 (Bentz & Collins, 2001), including longitudinal and transverse reinforcing. The various sections that were analysed are shown in Appendix N and include sections for each of the five different standards periods (pre-1930, 1930-59, 1960-79, 1980-99 and post-2000), and their variations for light and heavy claddings. Only one-storey perimeter foundations and their corresponding loads were modelled for this research. This resulted in eight different concrete perimeter sections being modelled, as the pre-1930 and 1930-59 periods did not have variations for cladding weight. Of the 59 concrete perimeter houses inspected (including the no liquefaction inspections) there were only five houses that were not one-storey, making it difficult to compare the field data collected with any two-storey foundation modelling.

Table 6.1. Distribution of floor levels above ground in inspected concrete perimeter foundation houses.

Approx. Floor Level Above Ground	# of inspected houses
<200mm	0
200-400mm	14
400-600mm	40
>600mm	4

The minimum requirements in terms of dimensions from the standards, as shown in Appendix A, could have been used to ensure a conservative estimate of section strength. However, as was found in the invasive inspections of Chapter 5, and the detailed inspections in Chapter 4, the majority of concrete perimeter beams are deeper than the minimum requirements in the standards. Table 6.1 shows the distribution of floor level heights in concrete perimeter foundation houses that were inspected for Chapter 4. This shows that the majority of concrete perimeter foundations have floor levels between 400-600mm above the ground, which corresponds to above ground perimeter wall heights of approximately 300-500mm, when the subfloor structure is taken into account. Adding on the depth that perimeter walls are cast into the ground, which the standards reviewed for this research require to be a minimum of between 200-300mm (which agrees with what was found in the invasive inspections in Chapter 5), gives a common constructed perimeter total wall depth range of 500-800mm. For modelling purposes, a representative perimeter depth of 600mm was used for section strengths. This was chosen as it was close to the smaller end of perimeter wall depths, so represents a conservative estimate of foundation performance, and it provides a good representation of the average concrete perimeter foundation height of houses in Christchurch. A typical unreinforced section size 600mm deep and 200mm wide was assumed to represent pre-1930 concrete perimeter sections for modelling purposes, given there were no standards to provide a required minimum.

Response-2000 was used to obtain cracking, yielding and ultimate moments for each different concrete perimeter cross section, as well as the ultimate axial tension load, for comparisons with lateral spreading loads. Section strengths were obtained for each different perimeter cross section for both positive (tension on the bottom) and negative (tension on the top) moments, which represent loads from the house acting down and loads from the foundation soils pushing up respectively. This was necessary as the range of soil profiles that were modelled gave bending moment profiles with both positive and negative moments. An example moment-curvature plot for the 1930-59 one-storey concrete perimeter section is shown in Figure 6.1.

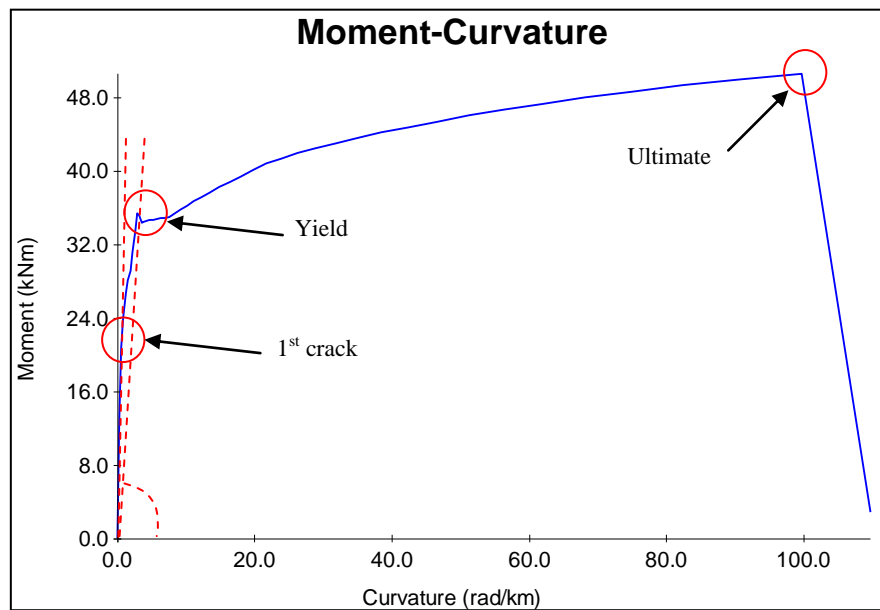


Figure 6.1. Typical moment-curvature relationship for a concrete perimeter section, showing locations of first cracking, yielding and ultimate failure and gross initial and secant stiffnesses. This particular plot is for a 1930-59, one-storey section.

6.2.2 Beam Modelling

Modelling of foundation beam elements was undertaken in the structural analysis package SAP2000 to provide moment demands produced by each loading scenario to compare to the concrete perimeter section capacities. The general beam model used for this modelling is shown in Figure 6.2. Soil springs are located at the nodes between each of the beam elements to represent the ground beneath, with the house weight is applied as a uniformly distributed load on top. The soil springs were degraded in stiffness by different degrees, over varying lengths, to represent a variety of loading scenarios.

The beam was modelled elastically, as more detailed inelastic modelling was outside the scope of this research. Elastic modelling was considered appropriate to compare to the section properties up until the yield capacity of the concrete perimeter sections is reached. This is because, due to the low reinforcing steel contents (or complete absence of reinforcing) in all sections modelled, the cracked section stiffness is similar to the initial elastic stiffness, and yield occurs with very little additional

curvature after first crack is reached. This results in the secant stiffness at yield also being similar to the initial elastic stiffness, as shown in Figure 6.1. The moment-curvature relationships for all sections are shown in Appendix O, and all had similarly shaped curves, with high cracking stiffnesses. The exception is the pre-1930 section, which has no reinforcing and reaches its ultimate capacity at the point of first crack. Because of this, the bending moments produced by the different loading scenarios were used to directly compare to the yield strengths of the concrete perimeter sections modelled. Using an elastic analysis meant that the resulting displacements were not useful. This is because the post-yield behaviour of the sections is very different. Once yield is reached, the section stiffness is much lower and large strains develop, and this behaviour is not represented by the elastic model. The displacements which are of interest for liquefaction and foundation damage are the large permanent displacements resulting from plastic deformation, which are not calculated by this model. These displacements are also outside the scope of this modelling.

A number of steps were required to conduct this beam modelling, which are outlined in this section.

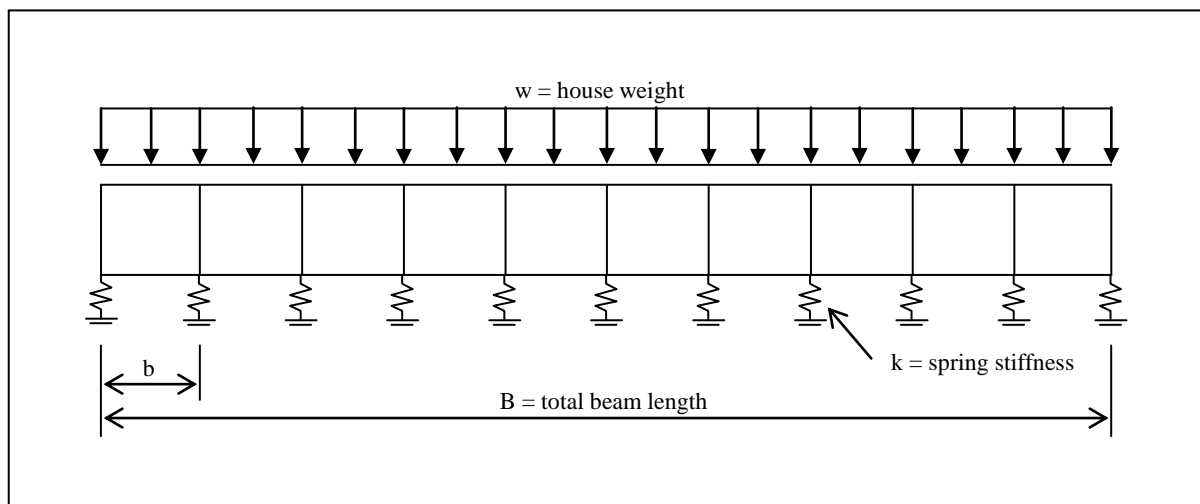


Figure 6.2. Schematic of the beam model used in SAP2000.

6.2.2.1 House weights

The first step was to calculate the loads that would be imparted to the foundations by the house superstructure above. These vary depending on the exterior wall cladding, the roofing material and the number of storeys. Total weights were calculated for each part of the house, including:

- Roof, both light and heavy claddings using weights given in NZS3604
- Roof Framing, assuming a standard timber member size of 100x50mm and density of 5.5kg/m^3
- Ceiling, including framing (again 100x50mm) and 10mm plasterboard
- Ground Floor, assuming 125x50mm timber with 20mm particle board on top
- External Walls, both light and heavy, again using weights from NZS3604 and including framing.

- Interior Partitions, with 10mm plasterboard either side and framing, assuming a representative 50m in length for the whole house.

The spacing of members was assumed for the different types of framing to allow a load distribution in kPa to be calculated in each case, which was then multiplied by the area of the house to give the total load. Example calculations are shown in Appendix P and were based on personal communications with Graeme Beattie, Principal Engineer at BRANZ.

The total weights for each part of the structure were then added up to give the total weight of the entire superstructure. This was then divided by the perimeter of the house to give the uniformly distributed load on the concrete perimeter. The whole weight of the house was divided onto the perimeter, including the interior walls and the whole floor. This was done to be conservative, and as other dead loads for internal items such as cabinetry and furniture, and live loads were not included in the calculations.

Table 6.2. Uniformly distributed load results for 12 different house plans, including average loads and standard deviation, for the different combinations of roof and wall cladding.

Area (m ²)	Perimeter (m)	A/P ratio	UDL (kN/m) for Heavy (H) or Light (L) roof/cladding			
			H/H	L/H	H/L	L/L
127.5	47	2.71	10.79	8.71	6.32	4.24
119	45	2.64	10.72	8.69	6.25	4.22
136	50	2.72	10.76	8.67	6.28	4.20
132	54	2.44	10.30	8.39	5.83	3.92
110	43	2.56	10.64	8.66	6.16	4.18
130	57	2.28	10.02	8.22	5.55	3.75
161.5	65	2.48	10.24	8.31	5.77	3.84
115	51	2.25	10.06	8.28	5.59	3.80
192	66	2.91	10.86	8.65	6.38	4.18
214	68	3.15	11.19	8.83	6.72	4.36
220	64	3.44	11.65	9.10	7.17	4.63
Average			10.66	8.59	6.18	4.12
Std. Dev.			0.4891	0.2650	0.4891	0.2650

The uniformly distributed loads acting on the concrete perimeter were calculated using the floor plans of twelve different houses that were inspected, with different floor plan shapes and sizes, to ensure a range of possibilities in terms of area and perimeter lengths were considered. The average uniformly distributed loads from these calculations were then used for the modelling in SAP2000. This is shown in Table 6.2, which shows the results for the different combinations of heavy and light roofing and heavy and light external cladding materials for the twelve different floor plans, and the average values that were used in the SAP2000 modelling. There is little variation in the results for the different floor plans, as shown by the standard deviations for each cladding combination. This shows that despite a

large range in house sizes and layouts, the actual load on the perimeter remains quite consistent, and is only significantly dependent on the types of roof and wall cladding.

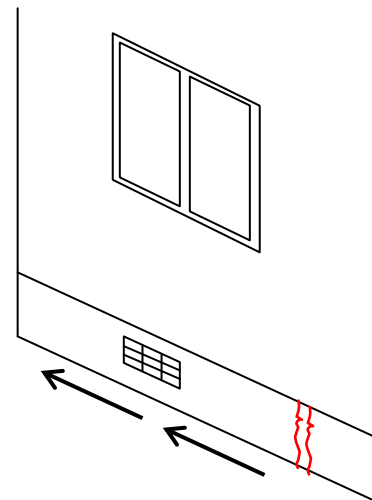
6.2.2.2 Foundation Load Cases

A number of foundation load cases were considered for modelling. They were created for concrete perimeter and slab-on-grade foundations based on observations made in the field during the detailed inspections of the types of loading that could reasonably occur during liquefaction and lateral spreading. They represent a variety of different possible loading scenarios for foundations subjected to liquefaction and lateral spreading effects, and combinations of the two. Although slab-on-grade foundations were not modelled for this research, it is useful to consider their unique load cases, along with those that are similar to concrete perimeter foundations. These are shown in the following sections.

6.2.2.2.1 Perimeter

1. Dragging (Lateral Spread)

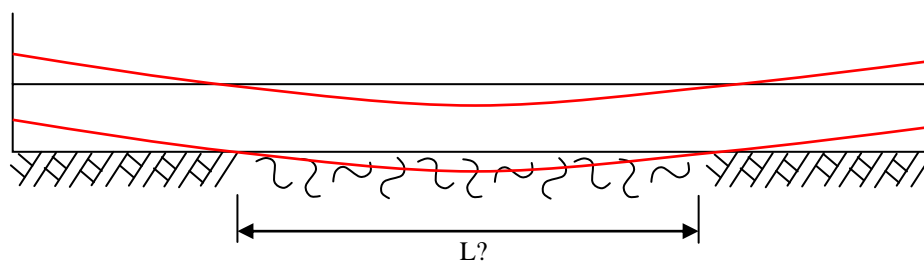
- The drag force caused by lateral spreading is influenced by the weight of the house acting downwards and the length of perimeter that is embedded in the ground. This drag force increases with the length of foundation over which it is applied. So what length of concrete perimeter must be subjected to a drag force (i.e. $1/3^{\text{rd}}$, $1/2$, $2/3^{\text{rds}}$) to cause different levels of damage such as cracking, yielding and failure.



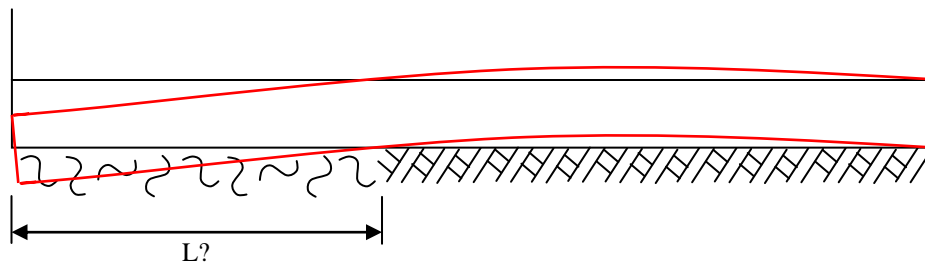
2. Loss of Bearing Capacity (Liquefaction)

What severity of liquefaction, corresponding to a length (L) and level of subsequent bearing loss in the soil is required under the foundation in each of the following loading cases to cause cracking, yield and ultimate failure. For liquefaction:

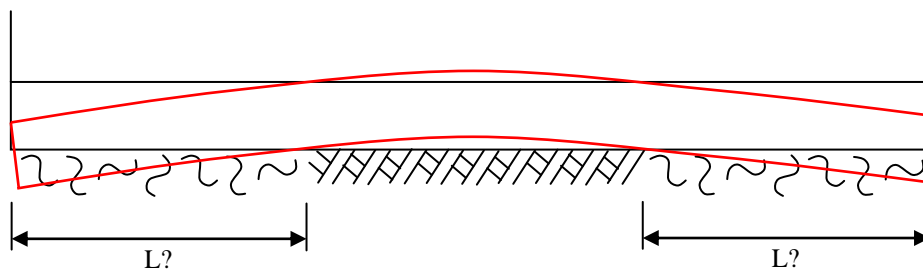
- a) In the middle:



b) At one end:

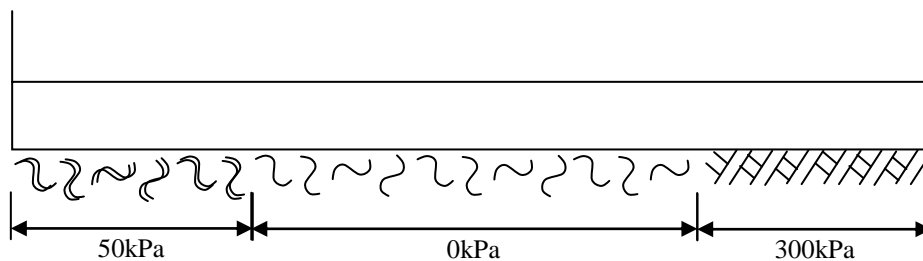


c) At both ends:



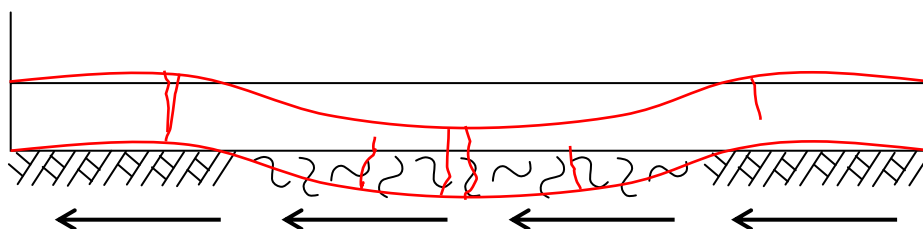
d) Non-uniform distributions:

What degree of bearing loss, combined with different lengths of each, will cause cracking, yield and ultimate failure in the foundation.

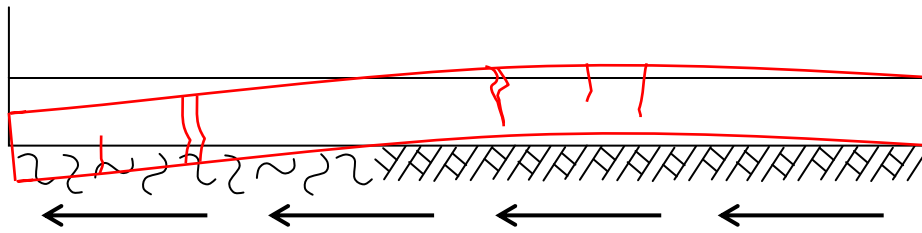


3. Dragging + Reduced Bearing (Lateral Spread + Liquefaction)

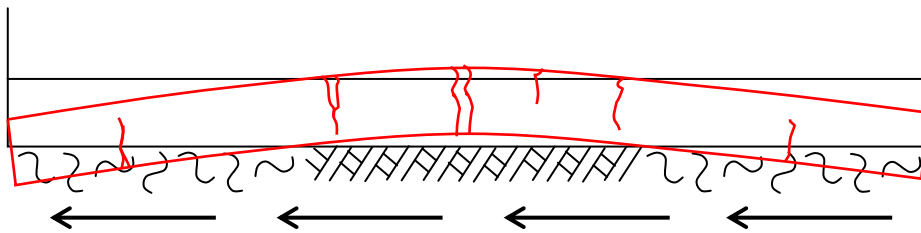
a) In middle:



b) At one end:



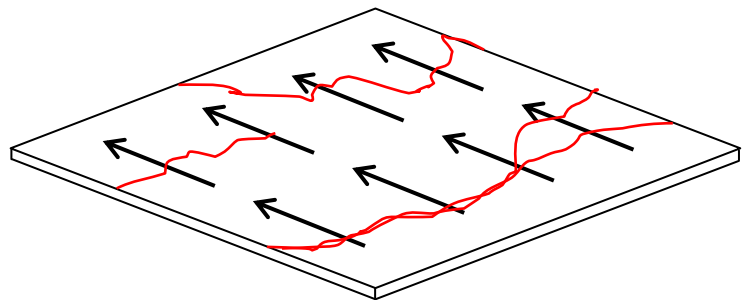
c) At both ends:



6.2.2.2.2 Slab

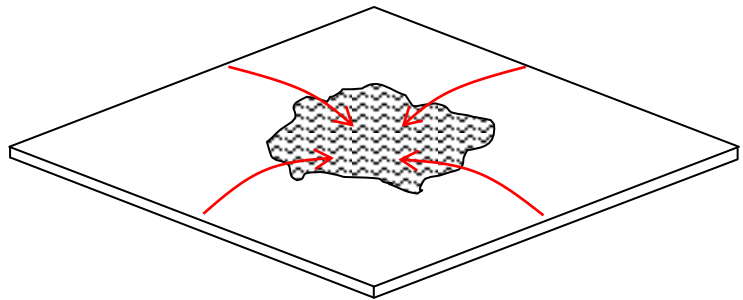
1. Dragging (Lateral Spread)

What width of slab must be subjected to a drag force (i.e. $1/3^{\text{rd}}$, $1/2$, $2/3^{\text{rds}}$) to cause different levels of damage such as cracking, yielding and failure.

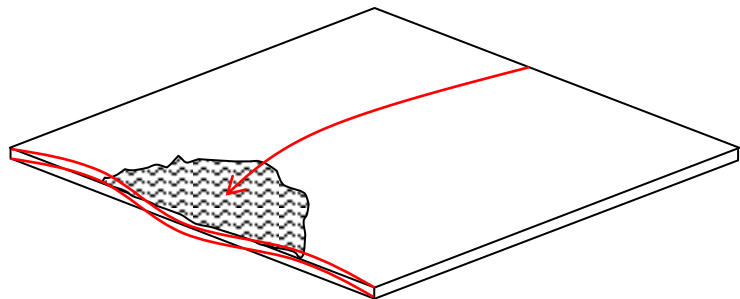


2. Loss of Bearing (Liquefaction)

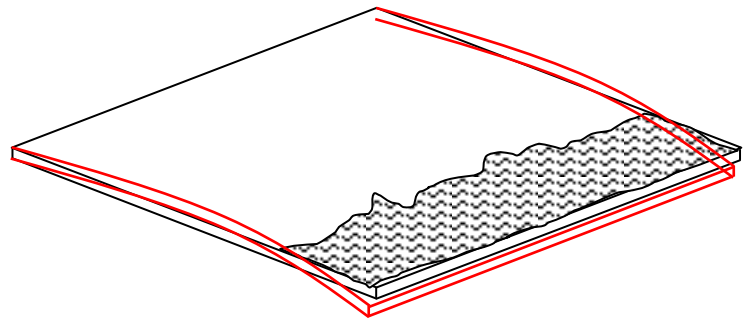
a) In the centre:



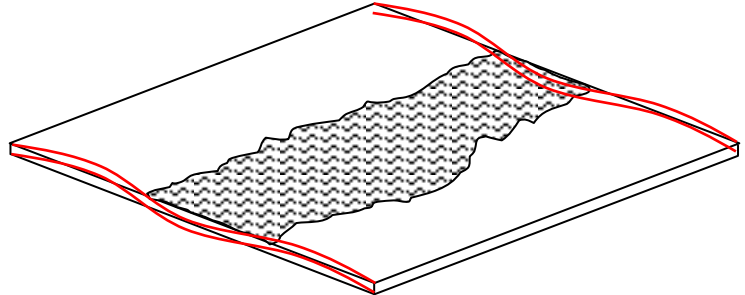
b) One side:



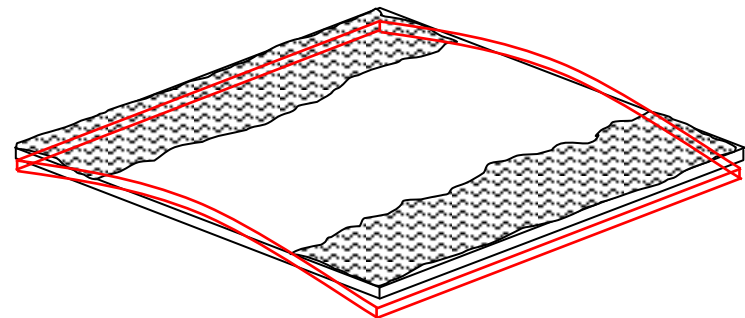
c) One end:



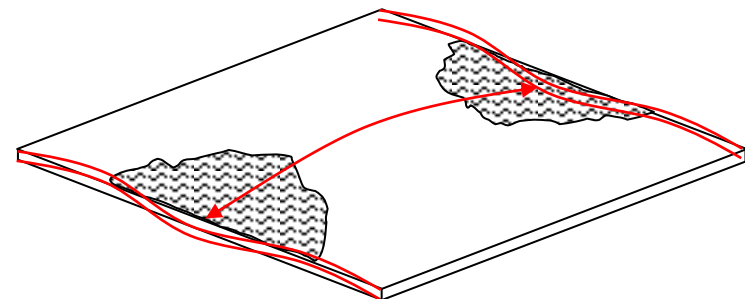
d) Middle:



e) Two ends:



f) Two sides:



The scenarios chosen for the modelling of perimeter foundations were Number 1, the dragging mode caused by lateral spreading, and Numbers 2.a), b) and c), the liquefaction-related loss of bearing in different locations over different lengths. The others were not used, as there are infinite combinations that could have been considered for 2.d), and Number 3, with the combinations of lateral spreading and liquefaction, over-complicates the issue. Using the more simplified loading patterns allows the results to be examined in more detail, and allows a more direct comparison back to the collected field data.

6.2.2.3 Soil Parameters

The next step was to calculate a value for k , the spring stiffness of the soil springs to be used in the SAP2000 beam model. This could only be very approximate, as the soils in Christchurch vary greatly, and the shallow nature of residential house foundations means that the extensive database of SPT and CPT records could not be used to represent soil conditions for the simplified model being used here.

Based on the above considerations the Approximate Flexible Method from Das (2004) was used, for foundations on sandy soils, for rectangular foundations. Equation 1, reproduced from Das (2004), gives a value for the coefficient of subgrade modulus, k , for a footing of length, L and width, B , based on the coefficient of subgrade reaction of a square footing of dimensions $B \times B$, $k_{(B \times B)}$.

$$k = \frac{k_{(B \times B)} \left(1 + 0.5 \frac{B}{L}\right)}{1.5} \quad (\text{equation. 1})$$

$$k_{B \times B} = k_{0.3} \left(\frac{B + 0.3}{2B}\right)^2 \quad (\text{equation. 2})$$

The value of $k_{(B \times B)}$ is given by equation 2, also reproduced from Das (2004), based on tabulated values for $k_{0.3}$ shown in Table 6.3. This shows typical values of subgrade reaction coefficient for different soil types. As the foundations are at or near the surface, the soil was deemed to be dry or moist, and as most top soil has a loose to medium density, the threshold value of $k_{0.3} = 25,000 \text{ kN/m}^3$ was chosen.

Table 6.3. Typical Subgrade Reaction Values, $k_{0.3}$ (reproduced from Das, 2004).

Soil type	k_1	
	MN/m ³	lb/in. ³
Dry or moist sand:		
Loose	8–25	30–90
Medium	25–125	90–450
Dense	125–375	450–1350
Saturated sand:		
Loose	10–15	35–55
Medium	35–40	125–145
Dense	130–150	475–550
Clay:		
Stiff	10–25	40–90
Very stiff	25–50	90–185
Hard	>50	>185

Using equations 1 and 2, this gave an approximate coefficient of subgrade modulus of $k = 26,302 \text{ kN/m}^3$. This is very close to the original $25,000 \text{ kN/m}^3$ selected from Table 6.3, as the foundation width of 0.2m used is close to the $B = 0.3 \text{ m}$ that the equations are based on, and so the value of $25,000 \text{ kN/m}^3$ was used due to the large approximation in this selected value. Multiplying this by the width of 0.2m gave a non-liquefied soil spring stiffness of $k = 5000 \text{ kN/m}^2$ for the footing being

considered. The calculations for this are shown in Appendix Q. To represent the loss of bearing capacity associated with soil liquefaction, the soil spring stiffness was reduced by certain percentages over a variety of lengths in SAP2000 to represent the loading cases described in Section 6.2.2.2.

6.2.2.4 SAP2000

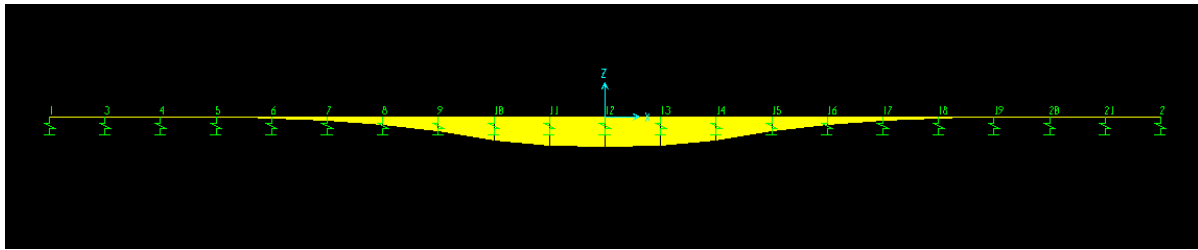
The beam modelling was conducted using the structural analysis package SAP2000 (CSi, 2010). Linear elastic beam elements 0.5m long were used to model a 10m long beam. The section layout from a standard 1960-79 perimeter with brick cladding was chosen arbitrarily as the beam section to input to the model, as the purpose of this modelling was to determine bending moments in the beam, and as linear elastic modelling was used, the actual beam section does not affect the moments calculated.

The soil was modelled using joint springs at each node in the beam (see Figure 6.2) to represent the soil springs with stiffness values set based on the calculated value of $k = 5000\text{kN/m}^2$ from above. Spring values were input based on the distance between the nodes in the model. For example, with a node spacing of 0.5m, $k_{\text{spring}} = 5000\text{kN/m}^2 \times 0.5\text{m} = 2500\text{kN/m}$.

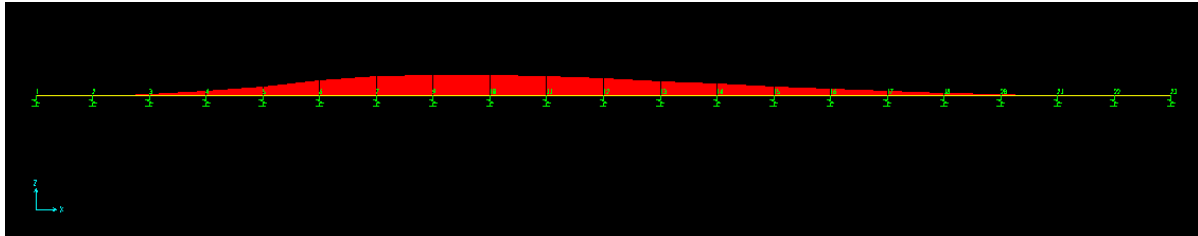
Four parameters were varied in the modelling. These were:

1. The house weight acting on the foundation with the four different combinations of roof and wall cladding weights shown in Section 6.2.2.1.
2. The location of bearing loss, between the centre, one end and both ends.
3. The level of reduction in bearing capacity. Four different reductions were used, with spring values 50%, 80%, 96% and '100%' lower than the non-liquefied stiffness of 5000kN/m^2 . These corresponded to soil stiffnesses of 2500kN/m^2 , 1000kN/m^2 , 200kN/m^2 and 0.002kN/m^2 respectively. This meant the joint springs in the model, for 0.5m element lengths, had values of 1250kN/m , 500kN/m , 100kN/m and 0.001kN/m respectively.
4. The total length of bearing loss in each case. The lengths used were 2m, 4m, 5m, 6m, 7m and 8m. This was to give values over the complete range of lengths, from a very small length to bearing loss over almost the whole foundation.

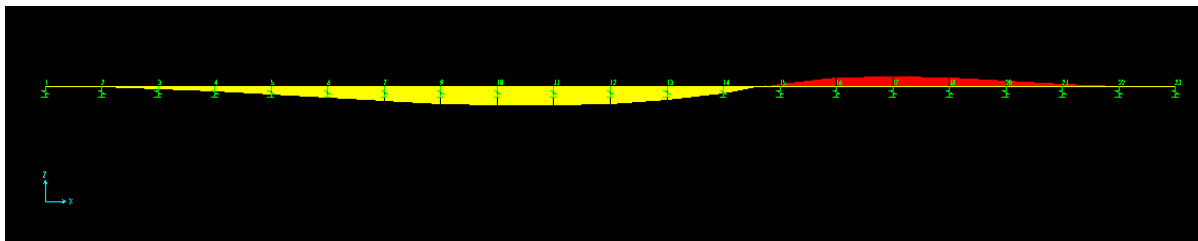
Figure 6.3 shows the SAP2000 model, with joint springs between each element. It also shows the shape of the bending moment diagrams for each loading case with different locations and lengths of bearing capacity loss. These images show the variation in location of the maximum bending moment and over what length it occurs. The sign of this maximum moment, or the side under tension also varies, which is why the section strengths had to be obtained for both negative and positive moments.



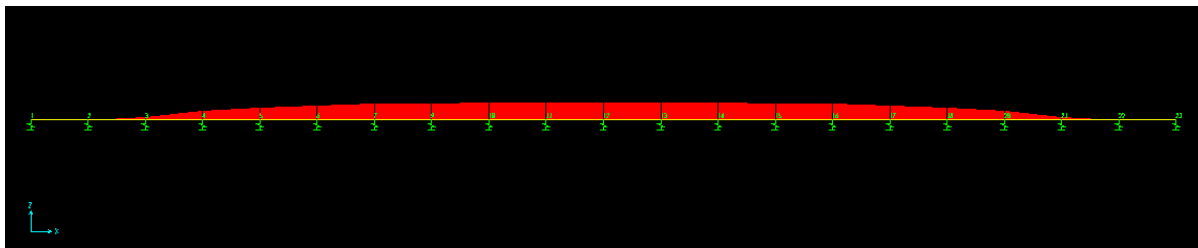
a) 50% bearing loss in centre, over 2m. $M_{\max} = 9.16\text{kNm}$



b) 80% bearing loss at left end, over 2m. $M_{\max} = -21.17\text{kNm}$



c) 50% bearing loss at left end, over 6m. $M_{\max} = 4.20\text{kNm}$



d) 80% bearing loss at both ends, 1m either end (total of 2m). $M_{\max} = -16.16\text{kNm}$

Figure 6.3. SAP2000 models showing spring and beam elements, and the variety of bending moment profiles obtained from the different loading combinations.

6.2.2.5 Sensitivity Checks

Sensitivity checks were carried out in SAP2000 to determine, first of all, the element length needed to ensure accurate results were given. Analyses were run for a 10m long beam that had the spring stiffness reduced in the centre by varying lengths (i.e. corresponding to case 3.a of the foundation load cases). The spring stiffness was reduced over lengths of 2m, 4m, 5m, 6m, 7m and 8m, first by 50%, down to $k = 2500\text{kN/m}$, and then by 96%, down to $k = 100\text{kN/m}$. The results of these analyses are shown in Table 6.4 and Table 6.5 for the 50% and 96% reductions respectively and in Figure 6.5. The bending moment diagrams for 50% bearing loss over 2m are also shown in Figure 6.4.

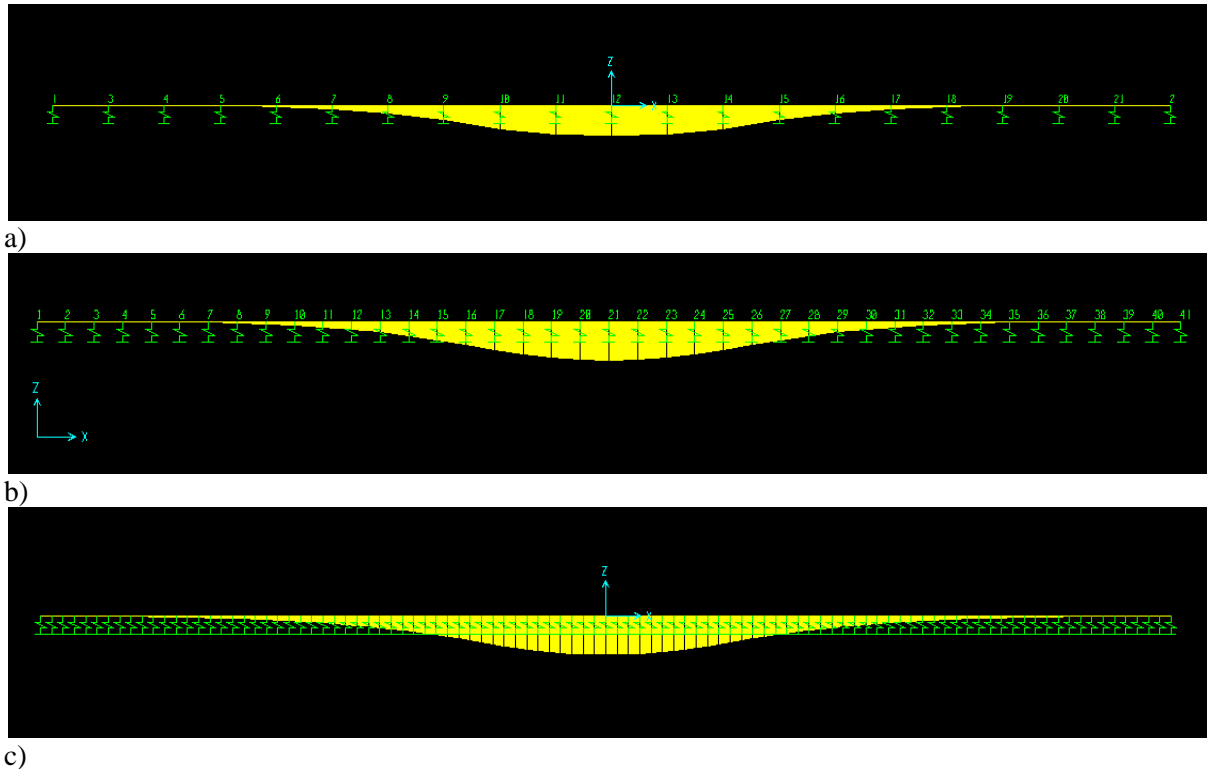


Figure 6.4. SAP2000 models showing bending moment diagrams for 2m of 50% reduced bearing in the centre, for a) 0.5m elements, b) 0.25m elements and c) 0.1m elements.

The results in the two tables show that the values obtained using the larger element size of 0.5m are quite close to those obtained using the much smaller element size of 0.1m, except for the reduction in spring stiffness over only 2m where the differences exceed 14% for both a 50% and 96% reduction in stiffness and over 8m for a 50% bearing loss. However, when viewed on the plot in Figure 6.5, the overall results are still very close. For the level of simplicity being used in this modelling, an element length of 0.5m returns suitably accurate results, and was used for all modelling in SAP2000. The relatively small amount of extra accuracy gained in using 0.25m or 0.1m element sizes is not justified in this modelling, given the approximate nature of other values such as the soil stiffness used.

Table 6.4. Maximum bending moments in a 10m beam and percentage differences for different element lengths for a 50% reduction in soil stiffness over the given lengths.

Length of bearing loss (m)	Max. Bending moment (kNm) for different element lengths			Percentage difference between different element lengths		
	0.1m	0.25m	0.5m	0.1-0.25m	0.1-0.5m	0.25-0.5m
2	8.01	8.46	9.16	5.6%	14.3%	8.2%
4	12.52	12.69	12.89	1.4%	3.0%	1.6%
5	13.35	13.36	13.30	0.1%	-0.4%	-0.5%
6	13.16	13.02	12.69	-1.1%	-3.6%	-2.6%
7	11.94	11.64	11.03	-2.5%	-7.6%	-5.2%
8	9.58	9.09	8.17	-5.1%	-14.7%	-10.2%

Table 6.5. Maximum bending moments in a 10m beam and percentage differences for different element lengths for a 96% reduction in soil stiffness over the given lengths.

Length of bearing loss (m)	Max. Bending moment (kNm) for different element lengths			Percentage difference between different element lengths		
	0.1m	0.25m	0.5m	0.1-0.25m	0.1-0.5m	0.25-0.5m
2	19.59	21.20	24.03	8.2%	22.7%	13.3%
4	42.51	44.43	47.79	4.5%	12.4%	7.5%
5	55.11	57.06	60.43	3.5%	9.7%	5.9%
6	67.29	69.08	72.14	2.7%	7.2%	4.4%
7	77.45	78.79	81.08	1.7%	4.7%	2.9%
8	83.13	83.53	84.14	0.5%	1.2%	0.7%

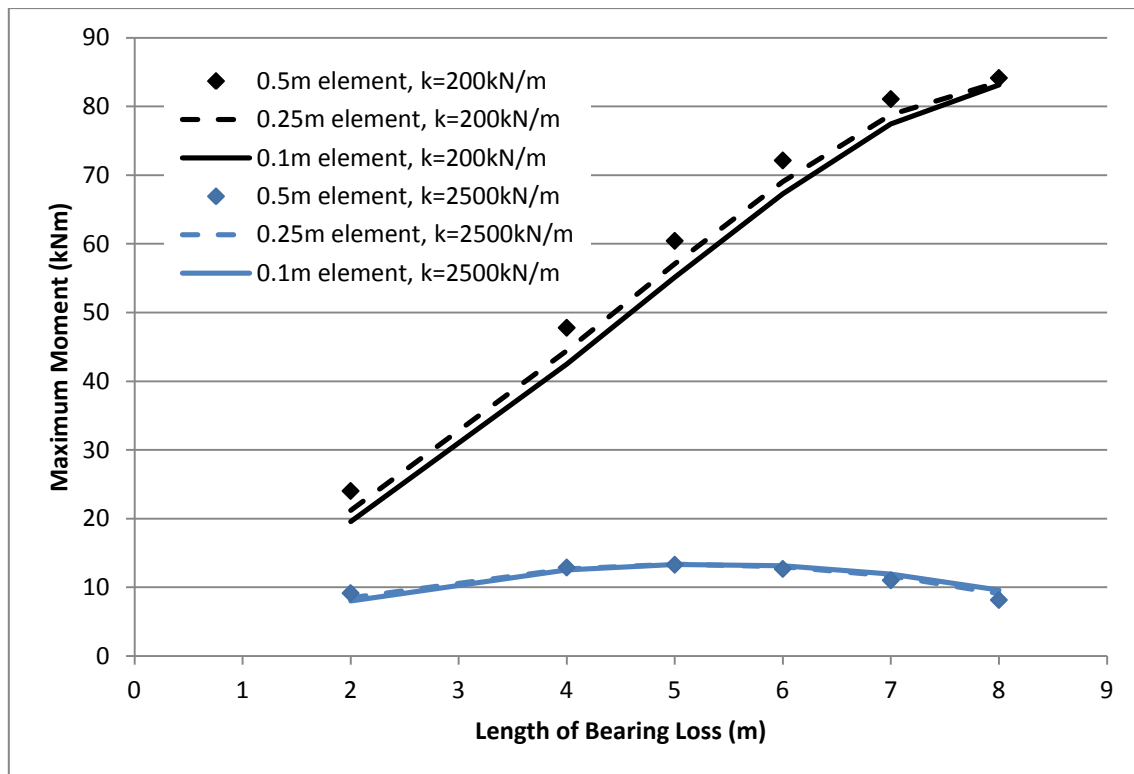


Figure 6.5. Maximum bending moments in a 10m beam with spring stiffness reduced over varying lengths by 50% (in blue) and 96% (in black) for different element lengths.

The next step was to ensure that the length of beam used didn't affect the results too much, to ensure any trends seen in the results were due to the soil parameters only, i.e. the length over which the bearing capacity (or spring stiffness in the model) is reduced, and the degree by which this is reduced. For this sensitivity check, total beam lengths of 10m, 15m and 20m were analysed under the same conditions. Again, reductions in stiffness of 50% and 96% over 2m, 4m, 5m, 6m, 7m and 8m. An element length of 0.5m was used in all cases. The results of these analyses are shown in Table 6.6 and Table 6.7, for 50% and 96% reductions in spring stiffness respectively, and in Figure 6.6.

Table 6.6. Maximum bending moments in beams of different lengths and the percentage differences between these for an element length of 0.5m and a 50% reduction in soil stiffness over the given lengths.

Length of bearing loss (m)	Max. Bending moment (kNm) for different beam lengths			Percentage difference between different beam lengths		
	10m	15m	20m	10-15m	10-20m	15-20m
2	9.16	8.08	7.92	-11.8%	-13.5%	-1.9%
4	12.89	11.30	10.95	-12.3%	-15.0%	-3.1%
5	13.30	11.86	11.42	-10.8%	-14.1%	-3.7%
6	12.69	11.83	11.33	-6.8%	-10.7%	-4.2%
7	11.03	11.32	10.79	2.6%	-2.2%	-4.7%
8	8.17	10.46	9.94	28.1%	21.7%	-5.0%

Table 6.7. Maximum bending moments in beams of different lengths and the percentage differences between these for an element length of 0.5m and a 96% reduction in soil stiffness over the given lengths.

Length of bearing loss (m)	Max. Bending moment (kNm) for different beam lengths			Percentage difference between different beam lengths		
	10m	15m	20m	10-15m	10-20m	15-20m
2	24.03	20.95	20.30	-12.8%	-15.5%	-3.1%
4	47.79	40.33	37.64	-15.6%	-21.2%	-6.7%
5	60.43	51.24	46.56	-15.2%	-23.0%	-9.1%
6	72.14	63.00	55.49	-12.7%	-23.1%	-11.9%
7	81.08	75.46	64.28	-6.9%	-20.7%	-14.8%
8	84.14	88.17	72.76	4.8%	-13.5%	-17.5%

The results for the 50% reduction in spring stiffness are very similar between the three beam lengths in Figure 6.6. However for a 96% reduction in stiffness they are somewhat varied. At 2m of stiffness reduction length, the difference is very small, but as the distance increases this difference grows significantly. However, there were no one-storey concrete perimeter foundations of any standard that had ultimate section strengths higher than 51kNm, and at this level the results are not as different as they are at higher bending moments.

The smaller beam length of 10m will be used for all analyses, as at smaller levels of spring stiffness reduction the results are not influenced by beam length, and at higher levels of reduction, the shorter beam gives more conservative estimates of foundation loads, and as 10m is a commonly seen length of walls across a house this will be used for all analyses.

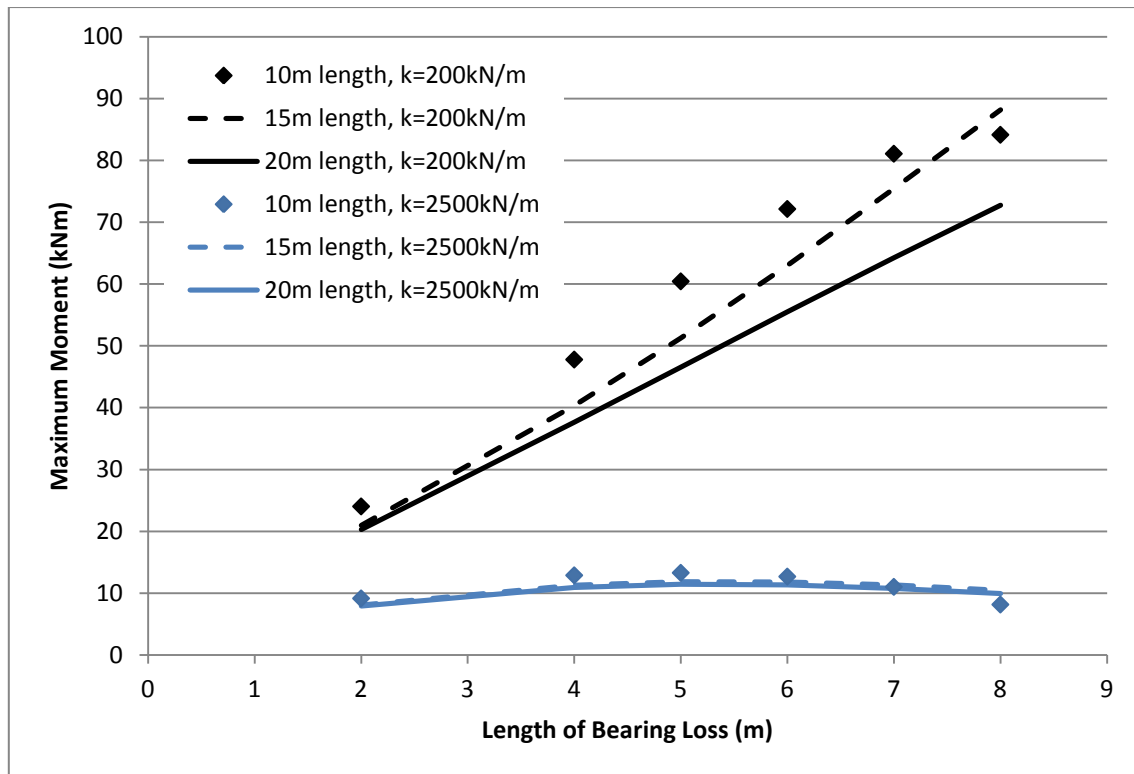


Figure 6.6. Maximum bending moments in 10m, 15m and 20m beams with spring stiffness reduced over varying lengths by 50% (in blue) and 96% (in black) with a 0.5m element length.

6.2.2.6 Lateral Spreading

The drag force on a foundation due to lateral spreading was calculated for the section shown in Figure 6.7.

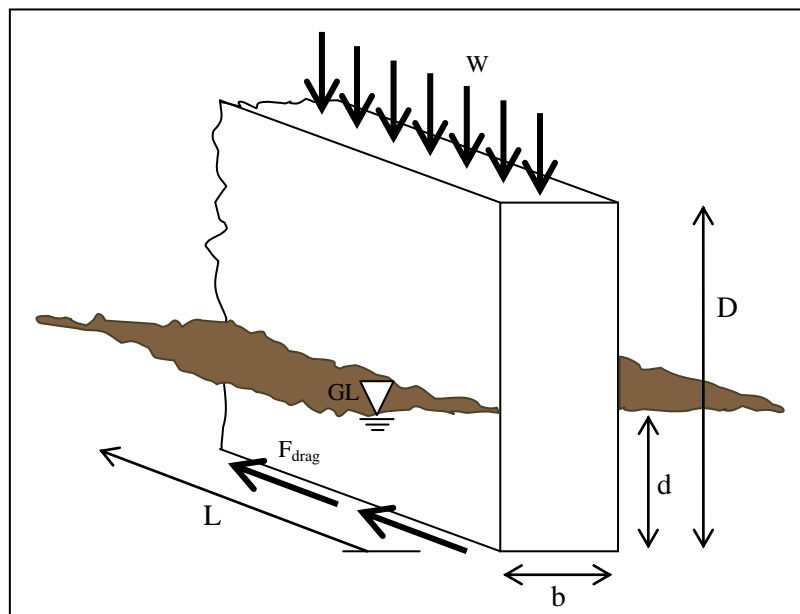


Figure 6.7. Schematic showing the key parameters for calculating the lateral spreading loads on the concrete perimeter foundation.

The following steps were used in determining the drag force from lateral spreading over a range of foundation lengths:

1. A critical state angle for the soil of $\phi'_{crit} = 30^\circ$ was assumed.
2. The self-weight of the concrete perimeter foundation was added to the house weights already calculated, based on a $D = 600\text{mm}$ deep foundation, $b = 200\text{mm}$ wide with a unit weight of 24kN/m^3
3. The drag force per unit length of the foundation was then calculated for each different house weight case (i.e. light or heavy roof and wall cladding) using the following equation:

$$\overline{Drag} = \overline{Drag\ stress} \times \text{Contact area}$$

$$\bar{F}_{drag} = \tau \times A$$

Where the drag stress per unit length, τ , is:

$$\tau = \sigma_N \cdot \tan \phi'_{crit}$$

$$\tau = \frac{w}{b} \cdot \tan \phi'_{crit}$$

For the uniformly distributed load, w and foundation width, b , (shown in Figure 6.7). The contact area of the foundation and soil that is dragging on the foundation is:

$$A = b + 2d$$

Where b is the foundation width and d is the depth of embedment. This gives the unit drag force from lateral spreading as:

$$\bar{F}_{drag} = \frac{w}{b} \cdot \tan \phi'_{crit} \times (b + 2d)$$

4. A standard embedment depth of $d = 300\text{mm}$ was used, based on observations from the invasive inspections of Chapter 4. This was the required embedment depth in the standards between 1935 and 1999, and is more conservative than using the reduced 200mm, introduced in NZS3604:1999, as the larger contact area means a higher drag force.
5. The total drag force over varying distances was then calculated from:

$$F_{drag} = \bar{F}_{drag} \times L$$

Where L is the length of foundation over which drag is occurring.

6. This process gave the axial drag force on the concrete perimeter foundation that represents lateral spreading over a variety of foundation lengths.

6.3 Results

6.3.1 Section Modelling

The results of the section analyses carried out with Response-2000 are shown in Table 6.8 and Table 6.9 for bending moments and axial capacities respectively. For the full moment curvature plots see Appendix O.

Table 6.8. Bending moment values for the different one-storey concrete perimeter sections modelled, showing first crack, first steel yield and ultimate capacities for pure bending.

Age	Cladding	House Load			Soil Load		
		1st Crack	1st Yield	Ultimate	Ultimate	1st Yield	1st Crack
Pre-1930	Any	12	(12)	12	12	(12)	12
1930-59	Any	21	35	51	51	35	21
1960-79	Light	15	18	27	20	13	13
	Heavy	22	34	50	50	34	13
1980-99	Light	15	18	27	20	13	13
	Heavy	23	34	50	29	18	14
post-2000	Light	15	17	27	27	18	15
	Heavy	25	34	50	29	18	15

The un-reinforced pre-1930 section has a very low ultimate strength, equal to its first cracking moment, due to the weak brittle nature of the section. Once the concrete has cracked in tension, the section has failed, as there is no longer a mechanism to resist bending moment actions.

For the light cladding section options for the latest three reinforced sections, the moment at first yield is very close to the initial cracking moment. This is due to the very small steel contents in these sections. Once the concrete has cracked, and the tension in the section is transferred to the tension steel, very little additional load is required to yield this steel. Because of this, and even for the heavy cladding sections, there is very little additional curvature between first crack and yielding, as shown in Figure 6.1, and the moment-curvature plots in Appendix O. As described in Section 6.2.2, this means that the section stiffness after cracking is close to the un-cracked elastic stiffness, which means that the bending moments from the elastic analyses performed in SAP2000 can be used to compare against the yielding capacities of the sections. The yielding moments of all sections post-1930 are between 65-70% of their ultimate moment capacities. This is also due to the small steel contents. Once the steel has yielded, it cannot resist large additional load before failure.

The heavy cladding section options of the later three reinforced sections have higher cracking moments due to increased section sizes, and yielding and ultimate moments that are almost double the light cladding options, due to the increased steel contents (see Appendix A for details).

The 1930-59 section has the highest strength of all those modelled. This is likely the result of a somewhat knee-jerk reaction to the 1931 Napier earthquake when the first housing standards were introduced in 1935, trying to make sure the damage seen in this event wasn't repeated. The strength is also the same for loading in both directions, as the section is a symmetrical rectangle. From 1960 onwards, the inverted T-shaped section has been used, resulting in different capacities for loading in the two directions. Also from 1960, the strength requirement for foundations of houses with light claddings was significantly reduced, in recognition of the lower vertical loads on the foundations in these houses. But, the strength of perimeter foundations for heavily cladded houses remained very similar.

From 1960, the capacities of the foundations for light and heavy claddings under downwards loading from the house superstructure have remained almost identical, despite minor changes to section sizes and reinforcement contents and layouts. The moment resistance to reverse loading has changed though, with the reverse moment capacities for heavy claddings significantly reduced after 1980. Between 1980 and 2000 the reverse moment capacities for light claddings were about 30% lower than the downwards capacities, due to the asymmetrical section layout, but after 2000 this has been brought back up so the two are equal.

The sections are designed like this because liquefaction is not considered in any of the standards. Under everyday conditions, the only direction of loading is downwards with gravity, and during an earthquake, upwards inertial loading is very unlikely, and are unlikely to be an issue for light structures such as houses anyway. However, liquefaction can cause significant actions in both directions, which is why it must be considered in the design of house foundations.

The axial capacities in Table 6.9 show a similar trend to the moment capacities above. The unreinforced pre-1930 section has a very low capacity, and fails in a brittle manner once the concrete cracks. Perimeter sections built between 1930-59 after the introduction of the first building standard have the largest axial capacity of any of the sections due to their higher steel content.

Table 6.9. Axial capacities for the different one-storey concrete perimeter sections modelled, for axial tension.

Age	Cladding	Axial Capacity (kN)
Pre-1930	Any	65.0
1930-59	Any	194.2
1960-79	Light	105.1
	Heavy	174.8
1980-99	Light	98.9
	Heavy	117.5
post-2000	Light	111.1
	Heavy	115.7

Between 1960-79 the strength of sections for light claddings was reduced by almost half, while the capacity of heavy cladding sections remained close to previous years. However after 1980, the axial capacity of heavy cladding concrete perimeters was also significantly reduced, due to a reduced longitudinal steel content. This corresponds to the lower reverse moment capacities of these sections seen in Table 6.8.

6.3.2 Beam Modelling (SAP2000)

The maximum bending moment values obtained from the SAP2000 beam analyses are shown here only to compare against the yield capacities of the various concrete perimeter sections, and identify over what length, for each degree of bearing loss, each section is likely to yield. Once the yield capacity of any particular section is reached, in reality, the demands on the sections will not continue to increase as shown in the plots in this section. Once each section yields, large strains will occur, which will be focussed at the plastic hinges formed at yield. This is due to the lower post-yield stiffness of the sections, as described in Section 6.2.2. The continually increasing moments with length of bearing loss are shown in the figures in this section to allow comparison with the different yield capacities of all the concrete perimeter sections that were modelled. The demands above yield capacity must be ignored when considering the behaviour of each section.

6.3.2.1 Bearing Loss in Centre

With bearing loss in the centre, the concrete perimeter beam is subjected to single positive curvature (i.e. tension is created on the bottom of the beam) and the bending moments are all positive. As the length of bearing loss increases, so does the maximum moment, and the length over which the moment demand occurs. This is shown in the bending moment diagrams in Appendix R:1.

Figure 6.8 and Figure 6.9 show the results of the analyses for heavy structures and light structures respectively. The results of the other analyses are shown in Appendix S:1. These figures show the demands for the four different degrees of bearing loss over the range of lengths modelled, with the yield capacities of the relevant concrete perimeter sections also shown.

The demands created by a 50% reduction in bearing capacity ($k = 2500\text{kN/m}^2$) are quite low, below 20kNm for the heavy cladding (Figure 6.8) and below 10kNm for light cladding (Figure 6.9). The demands increase somewhat with another 30% drop in bearing capacity to $k = 1000\text{kN/m}^2$, but a larger proportional increase is seen when the extra 16% loss is included for $k = 200\text{kN/m}^2$ and further to 100% bearing loss. At lower levels of bearing loss, the results do not depend hugely on the length over which this bearing loss occurs, whereas for 96% and 100% loss, the demands increase quickly with increased length, quickly exceeding the capacities of all foundation sections in heavy structures. This is because at lower levels of bearing loss, the soil is still providing a significant amount of support along the full length of the foundation beam, reducing the demands. However once the

bearing loss nears 100%, the span which is subjected to the bearing loss basically becomes simply supported at each end, and as the length of bearing loss increases, so does the moment demand.

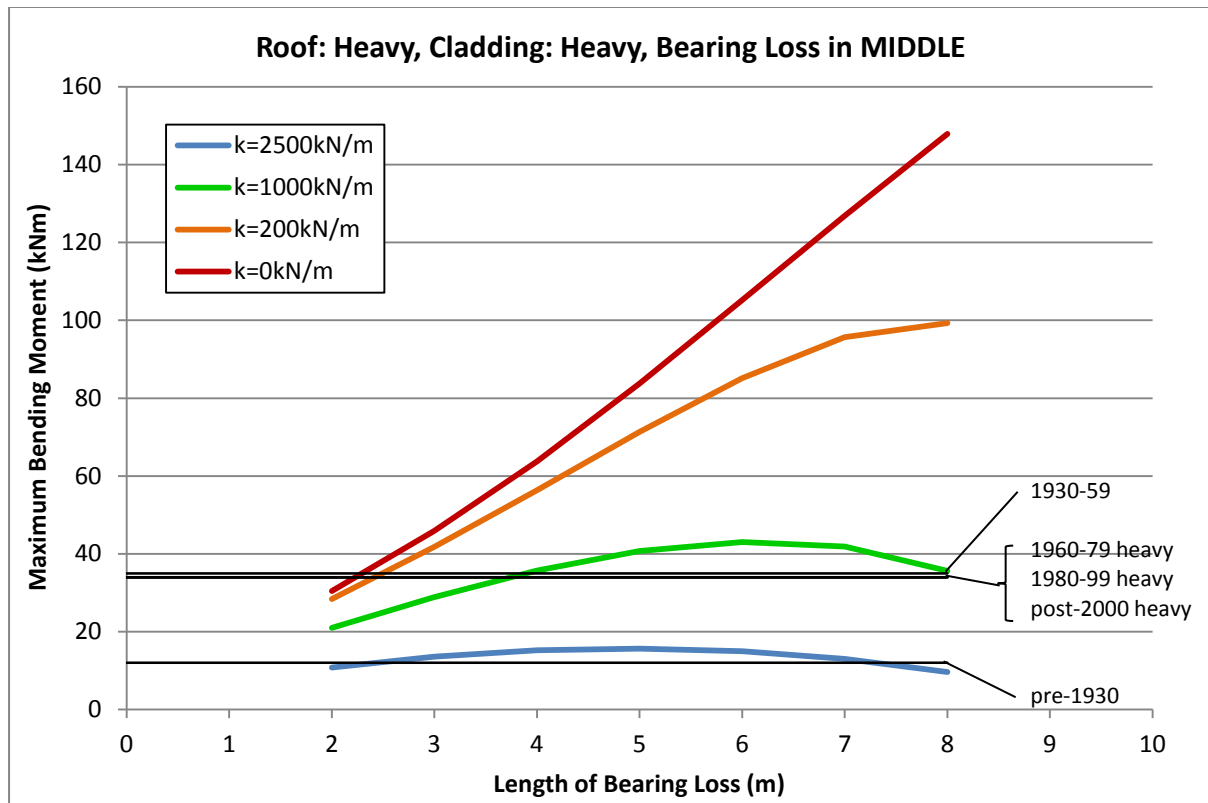


Figure 6.8. Moment demands for heavy roof and cladding for different levels and lengths of bearing loss in centre. Yield capacities of the corresponding perimeter sections are also shown.

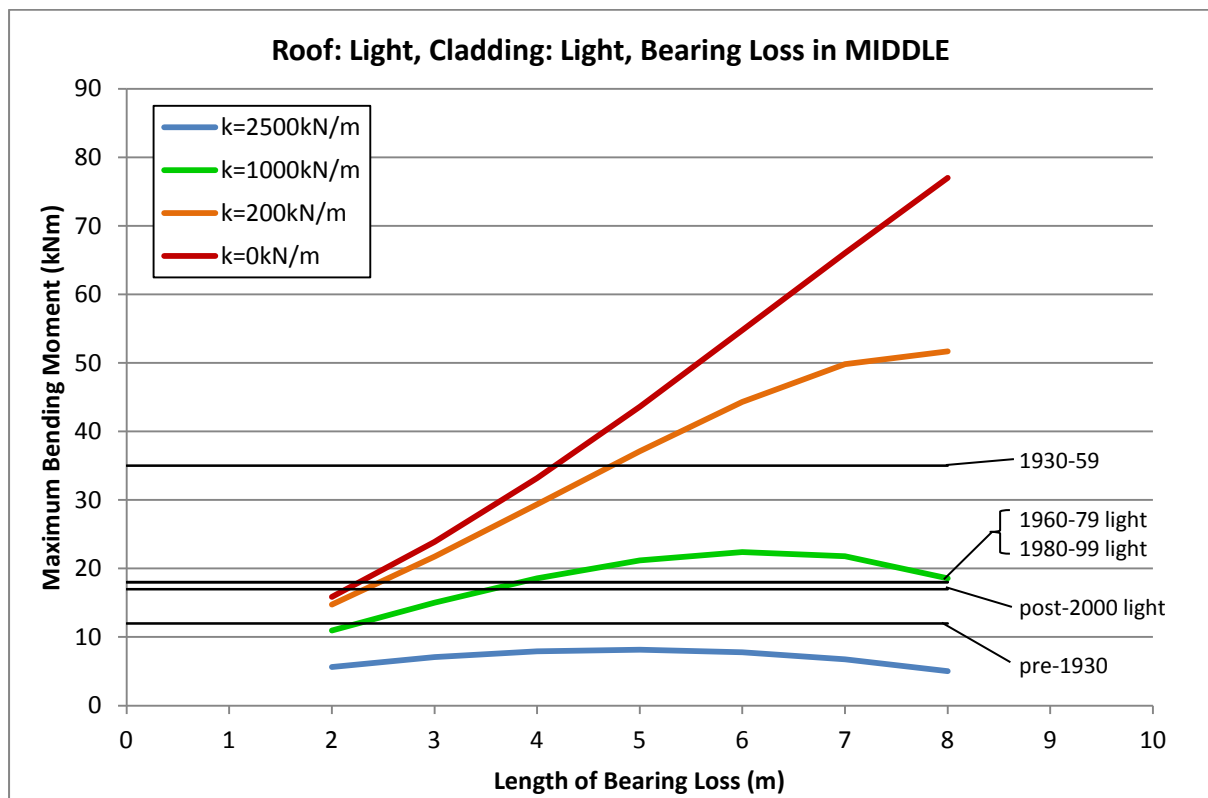


Figure 6.9. Moment demands for light roof and cladding for different levels and lengths of bearing loss in centre. Yield capacities of the corresponding perimeter sections are also shown.

For heavy claddings, as shown in Figure 6.8 and in Appendix S.1, Figure A. 129, the pre-1930 section performed poorly, and will fail under all levels of bearing loss except for 50% loss over less than 3m and more than 7m. The newer four sections performed better, and were able to resist 80% bearing loss up to 4m before yielding. However once the bearing loss was near to 100%, yield of these sections is expected with 2-2.5m of bearing loss. The results for heavy and light roofs were very similar for heavy claddings.

For light claddings the demands were significantly lower. This meant that the 1930-59 section, which does not change for cladding type, performed better. It will not yield under any length of 80% bearing loss, but will still yield after more than 4.5m of 96% bearing loss or 4m of 100% bearing loss with a light roof (Figure 6.9). With a heavy roof, these lengths reduce to 3.7m and 3.3m for 96% and 100% bearing loss respectively (Appendix S.1, Figure A. 132). The three newest foundations do not get any benefit from the reduced demands of a lighter structure, as their strengths are also reduced. This has resulted in almost identical performance with a light structure for these three sections when compared to heavy claddings. For light claddings with heavy roofs (Appendix S.1, Figure A. 132), their performance is worse, with 80% bearing loss over 2.5-3m causing yielding. The pre-1930 section still performs poorly, although with a light cladding it can resist 50% bearing loss over any length.

6.3.2.2 Bearing Loss at One End

Bearing loss at one end subjects the concrete perimeter beam to a range of loading patterns, depending on the length and degree of bearing loss. For smaller lengths of bearing loss, the beam is subjected to single reverse curvature (i.e. tension on the top), as the section where bearing loss has occurred is acting like a cantilever. However, once the length of bearing loss becomes long enough, the beam is subjected to double curvature. Positive curvature occurs along the section with bearing loss, as the soil is still providing some level of support. Negative curvature occurs over the beam where bearing capacity is still at 100%. Once there is 7-8m of bearing loss, the moment demand is almost all positive. These trends are shown in Appendix R.2.

Overall, the maximum moment demands for this loading situation are negative. At longer lengths of bearing loss the positive moments are larger, however these are still smaller than the maximum negative moments created under the same percentage of bearing loss for shorter lengths.

Figure 6.10 and Figure 6.11 show that at the lower percentages of bearing loss (50% and 80%) the shorter lengths are more critical. The largest moment is at 3m of bearing loss, and causes significant moments over a large portion of the beam. When the bearing loss is over a greater length, the positive moments have a larger influence. The double curvature reduces the negative moments, and the majority of the beam is acting more as a beam with a lower bearing capacity overall, with extra support at one end, than a beam that has lost bearing support. The peak is not large however, and the

moment demands for 50% and 80% bearing loss are quite flat, similar to the case of bearing loss in the centre. Particularly for lighter cladding (Figure 6.11), the moment demands do not depend heavily on the length of bearing loss.

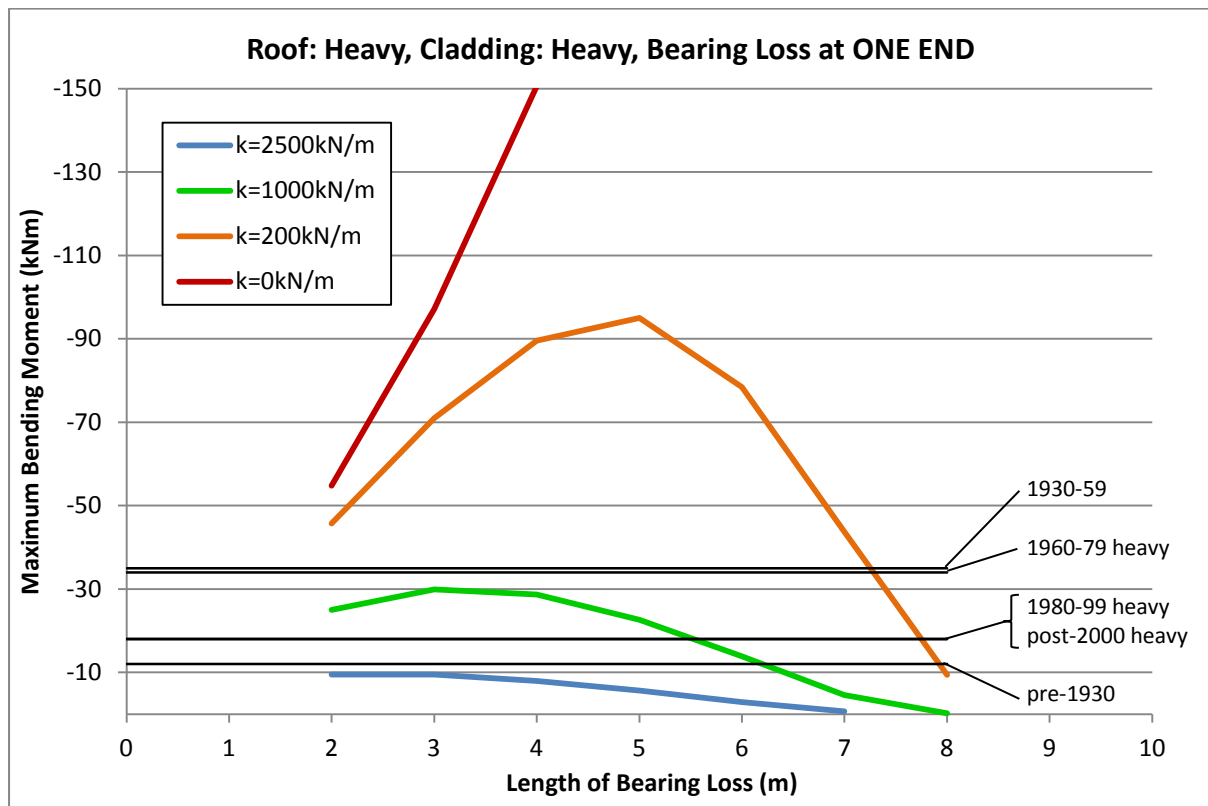


Figure 6.10. Moment demands for heavy roof and heavy cladding for different levels and lengths of bearing loss at one end. Yield capacities of the corresponding perimeter sections are also shown.

For 96% bearing loss, the largest moment demand occurs at 5m of bearing loss. The larger reduction in bearing capacity does not provide the same level of support for the rest of the beam. At 100% bearing loss the beam behaves very differently. As the beam receives no extra support from the length where bearing has been lost, it is effectively a cantilever beam, increasing in length. As the length increases so too does the moment demand. This is an extreme case and unlikely to occur in reality, but is shown here for comparison.

For heavy claddings, shown in Figure 6.10, the moment demands for 96% and 100% bearing loss exceed all section yield capacities for almost all lengths. The exception is for the 1930-59 and 1960-79 sections which can resist 96% bearing loss over more than 7.3m without yielding, due to the sharp drop-off in demand from 6m length. Although the pre-1930 section exceeds the 50% reduction demands, these are small due to the influence of double curvature already mentioned, and the performance of these old foundations is still very poor. The 1980-99 and post-2000 sections can resist 50% bearing loss and more than 5.5m of 80% bearing loss without yielding, which also can be seen as poor performance. This is due to their lower section capacities for the negative moment demands from this loading scenario, as shown in Table 6.8.

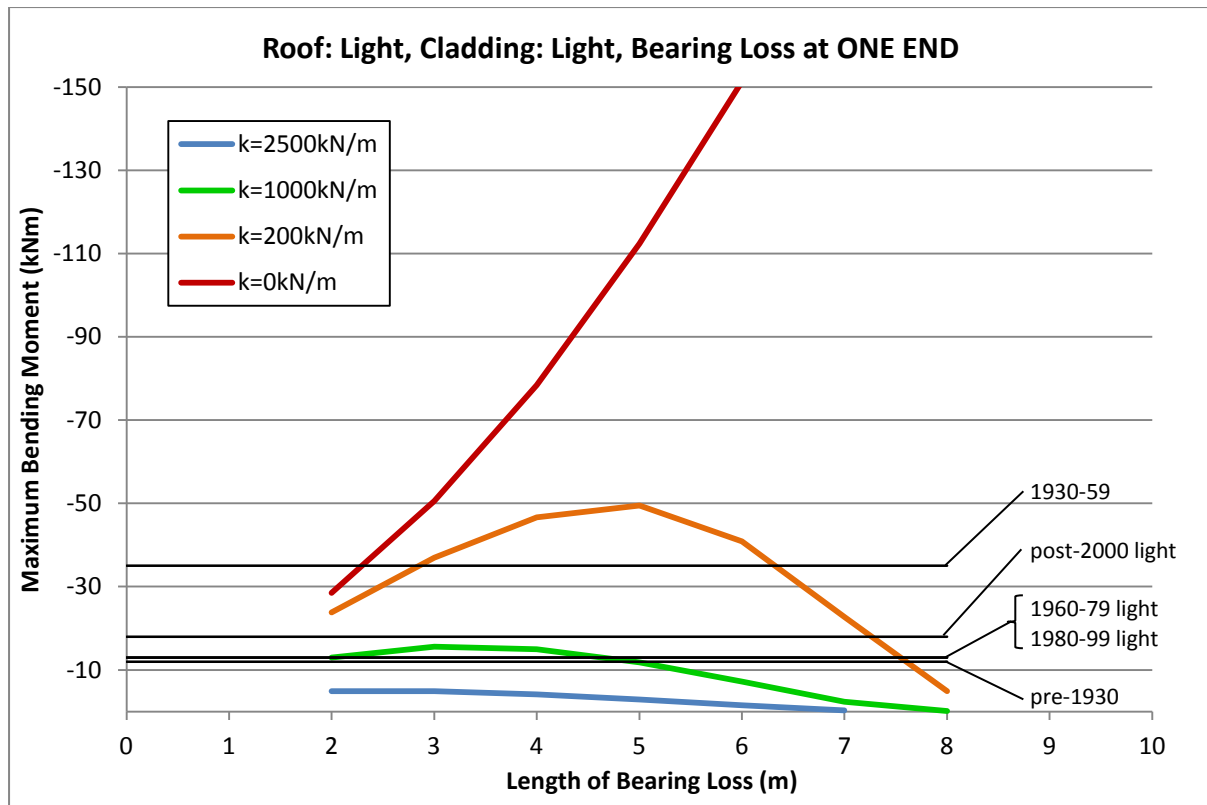


Figure 6.11. Moment demands for light roof and light cladding for different levels and lengths of bearing loss at one end. Yield capacities of the corresponding perimeter sections are also shown.

For light claddings, shown in Figure 6.11, the loads are significantly smaller. This is the same as with bearing loss in the centre, except for 100% bearing loss, which behaves differently due to the nature of the cantilever loading. For light roofs (Figure 6.11) the 1930-59 section yield capacity exceeds the demand from 96% bearing loss for less than 3m and more than 6.5m, but for heavy roofs (shown in Appendix S.2, Figure A. 136) the section can only resist less than 2m and more than 7m of 95% loss without yielding. The overall performance of the post-2000 section improves so that it can resist 80% bearing loss over any length without yielding. This is because for negative moment demand, its reverse yield capacity does not change for light claddings (see Table 6.8), so relatively, it performs better under the reduced moment demands of light cladding. The performance of 1960-79 and 1980-99 sections is somewhat better, due to the reduced demands for light claddings, however, their reduced yield capacities for light claddings means they perform almost as badly as the pre-1930 section, which is still considered to perform poorly.

6.3.2.3 Bearing Loss at Both Ends

Bearing loss at both ends of the concrete perimeter foundation beam causes similar bending moment patterns as bearing loss in the centre, only with negative bending moments instead of positive ones. The beam is subjected to single reverse curvature, and the maximum moment demands are similar in value to those for bearing loss in the centre. The distribution of these moments however is actually the other way around from bearing loss in the centre. For 2m of bearing loss (1m at each end), the

moment demand is small, but spread over most of the beam, whereas for bearing loss over 8m, the loads are large but over a smaller area. These patterns are shown in Appendix R.3.

For heavy claddings the results were very similar to bearing loss in the centre. As can be seen in Figure 6.12 for heavy roofs, all section yield capacities were exceeded by more than 4m of 80% bearing loss, which is only increased to 4.5m for light roofs (Appendix S.3). Due to the negative moment demands, the 1980-99 and post-2000 sections performed significantly worse than for bearing loss in the centre, due to their lower yield capacities for reverse loading. These two sections can only just resist 50% bearing loss, and will yield under 2-2.5m of 80% bearing loss, depending whether the roof is heavy or light. The pre-1930 section performs similarly to bearing loss in the centre, hovering around the 50% bearing loss demands.

For light claddings, shown in Figure 6.13, the 1930-59 section will yield with more than 3.5-4.5m of 96% bearing loss for heavy or light roofs. The post-2000 section again performs better with light claddings as its reverse strength is not reduced, allowing it to resist up to 4m of 80% bearing loss for light roofs before yielding. This is reduced to 3m for heavy roofs. The 1960-79 section performs worse for light claddings, due to its reduced strength, making it the same as the 1980-99 and pre-1930 sections. These three sections will all yield with just 2.5m of 80% bearing loss for light roofs, which reduces to 2m for heavy roofs.

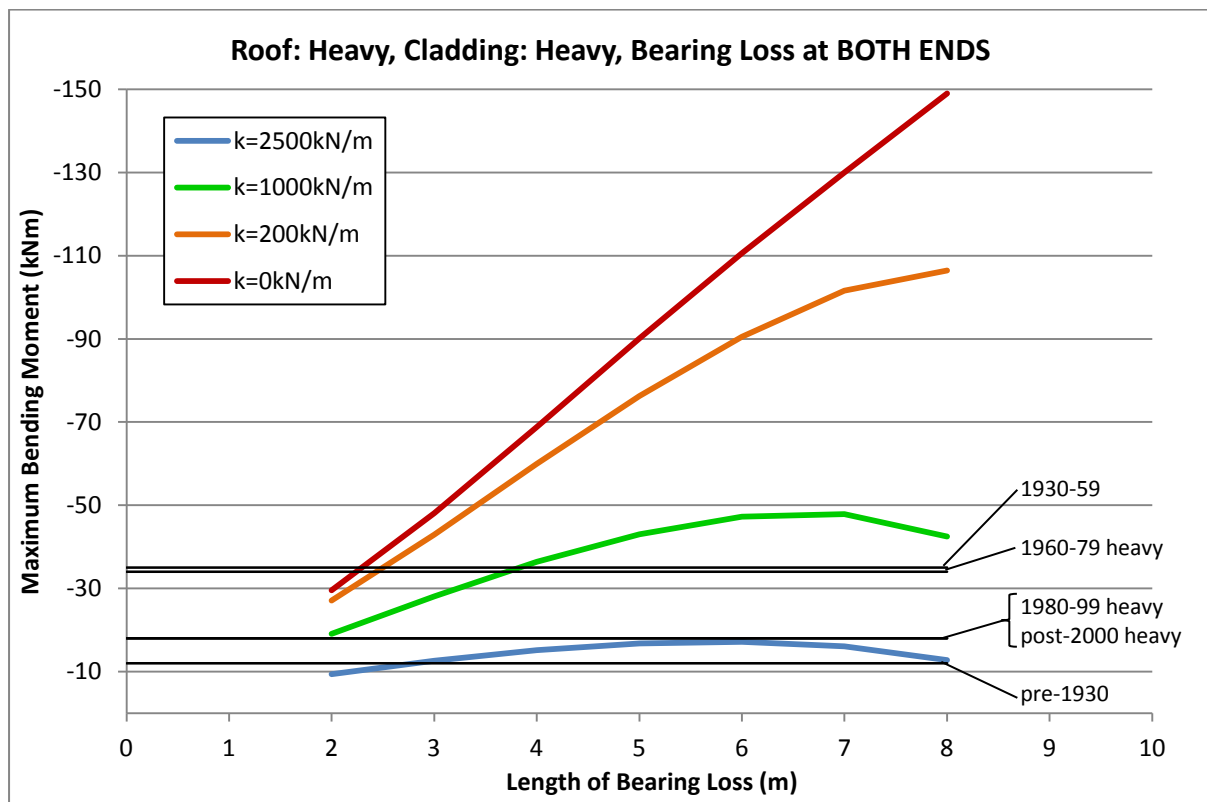


Figure 6.12. Moment demands for heavy roof and cladding for different levels and lengths of bearing loss at both ends. Yield capacities of the corresponding perimeter sections are also shown.

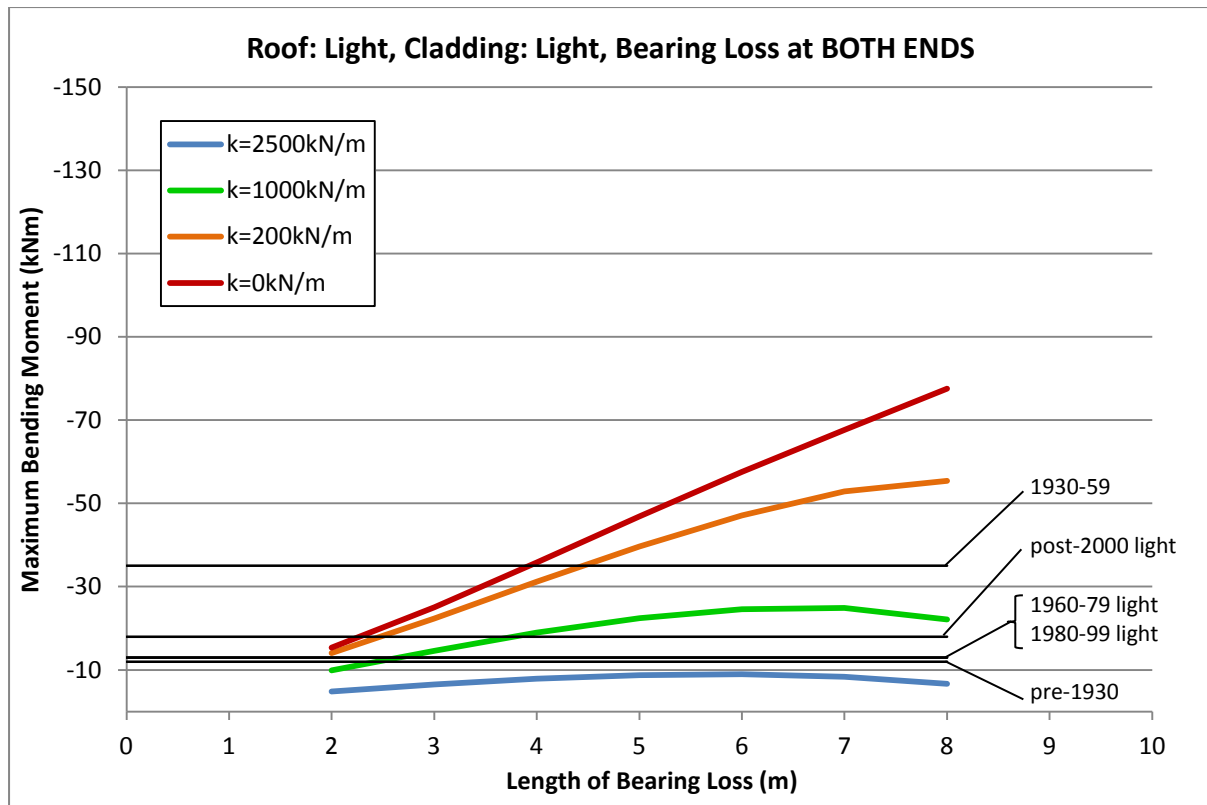


Figure 6.13. Moment demands for light roof and cladding for different levels and lengths of bearing loss at both ends. Yield capacities of the corresponding perimeter sections are also shown.

6.3.3 Lateral Spreading

The lateral spreading demands, represented with drag force along the foundation, increased linearly with increasing length. This is because the drag force is directly related to the length of contact. The demands for all cladding combinations are shown in Table 6.10, and compared against the relevant section strengths in Figure 6.14 and Figure 6.15 for heavy and light wall claddings respectively.

Table 6.10. Axial drag force on concrete perimeter foundations due to Lateral spreading in kN for the different combinations of roof and cladding materials.

Roof:	Light	Heavy	Light	Heavy
Cladding:	Heavy	Heavy	Light	Light
0	0.0	0.0	0.0	0.0
1	33.1	39.2	20.1	26.2
2	66.3	78.4	40.3	52.4
3	99.4	117.6	60.4	78.6
4	132.6	156.8	80.6	104.8
5	165.7	196.0	100.7	131.1
6	198.8	235.2	120.9	157.3
7	232.0	274.4	141.0	183.5
8	265.1	313.6	161.2	209.7
9	298.3	352.8	181.3	235.9
10	331.4	392.0	201.5	262.1

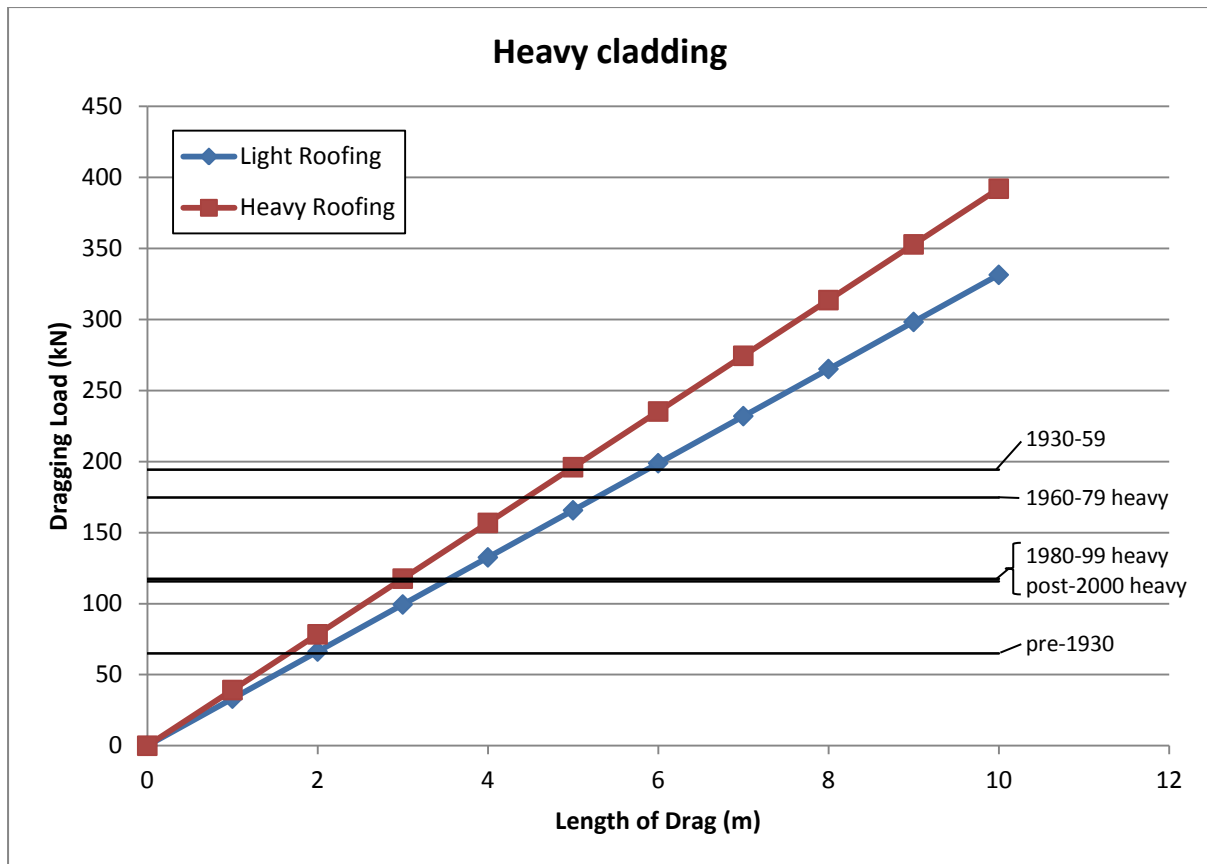


Figure 6.14. Axial tension loads caused by dragging along foundation over different lengths for heavy cladding, showing results for light and heavy roofing, with axial capacities of corresponding sections.

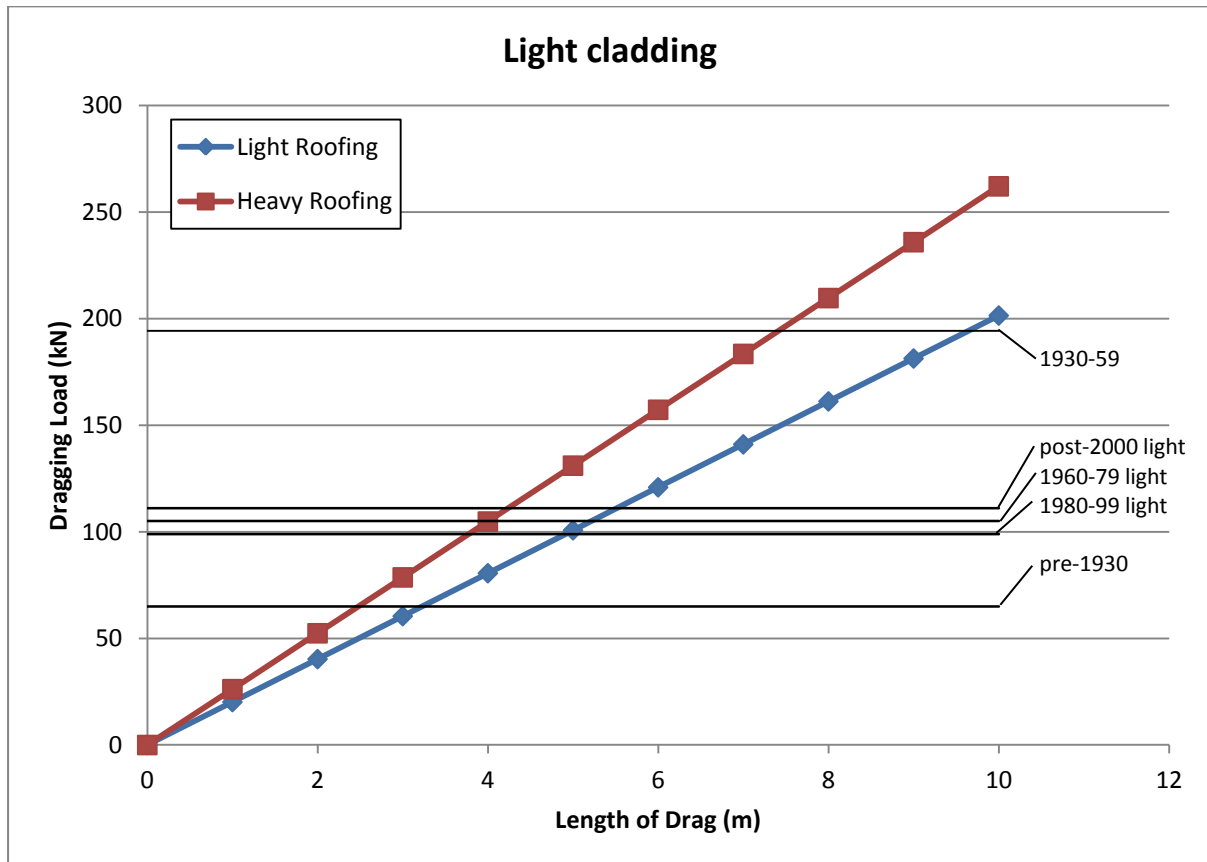


Figure 6.15. Axial tension loads caused by dragging along foundation over different lengths for light cladding, showing results for light and heavy roofing, with axial capacities of corresponding sections.

Heavy cladding, shown in Figure 6.14, produced demands 130kN higher than light cladding (Figure 6.15) over 10m of drag, with the roof cladding changing results by 60kN in each case, again at 10m of drag. This only caused a 15% change between light and heavy roofs for heavy cladding, but caused a 23% change for light cladding.

For heavy cladding in Figure 6.14, the pre-1930 section fails after 1.5-2m of drag. For the 1980-99 and post-2000 sections this increases to 3-3.5m of drag, and the 1930-59 and 1960-79 sections, which have a similar performance, can resist between 4.5-6m of drag. For the 1930-59 section, which has the highest axial capacity, having a heavy roof with heavy cladding means the section would fail at 5m, 1m earlier than for a light roof and heavy cladding, where failure occurs with 6m of drag, at a load of 200kN.

For light cladding, shown in Figure 6.15, the pre-1930 and 1930-59 sections performed better, as their design strength is not reduced based on cladding type. However, these two sections are still expected to fail under 4-5m and 7.5-9.5m of drag respectively. The newest three sections all have very similar capacity, due to reduced strengths for light claddings for all three. They are all expected to fail between 4-5.5m of drag depending on the section and roof cladding. This corresponds to a slight improvement for 1980-99 and post-2000 sections from the heavy cladding case, as the demands under light cladding have reduced more than their axial capacities. The 1960-79 section performance is reduced, as its strength is reduced the most between heavy and light claddings. Heavy roof cladding has a larger effect when the wall cladding is light, causing the 1930-59 section to fail 2m earlier at 200kN than with light roof cladding.

The axial capacities of the sections shown in Figure 6.14 and Figure 6.15 are maximum tensile capacities. Once these are reached, the section has failed. Significant damage to the concrete perimeter sections can occur at lower axial loads, as the concrete cracks and the reinforcing begins to yield. In addition, these maximum tensile capacities can significantly reduce if the section is also subject to moment demands causing additional tension in either the top or bottom of the section, due to the moment and axial interaction for the concrete sections. This is shown in Figure 6.16, where the axial tension capacity of the section reduces from the maximum possible as the moment demand on the section increases. This is almost inevitably the case with foundations suffering from lateral spreading, as to have lateral spreading, the underlying soils are usually liquefied, which is very likely to cause moments like those shown in Section 6.3.2 above.

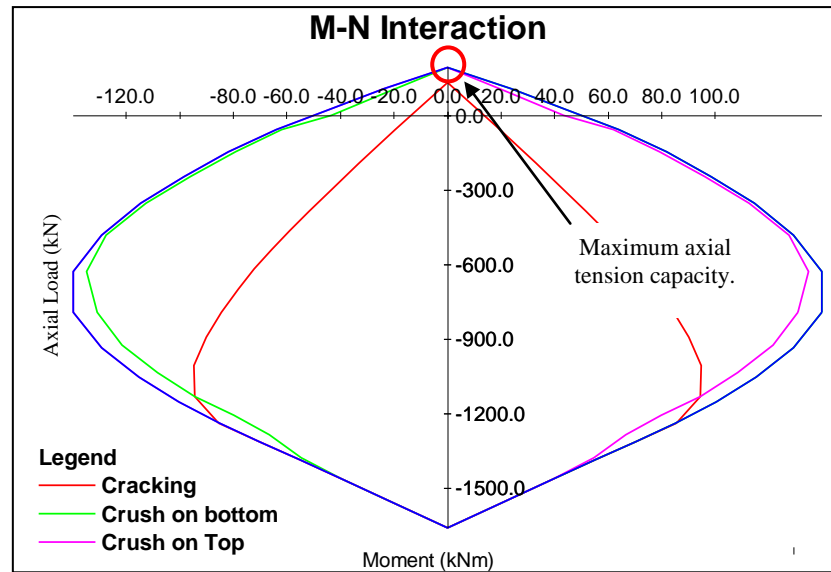


Figure 6.16. Example Moment-Axial interaction plot for the 1930-59 light cladding concrete perimeter section. Negative axial load is compression.

6.4 Discussion

There was a variety in performance for concrete perimeter foundations found in this modelling, due to the differences in section strength for foundations built under the different construction standards, and the variations in loading due to exterior wall and roof cladding materials.

Pre-1930s foundations were found to perform very poorly. Their section strength is very low, less than half that of the sections for light claddings in newer houses, and due to the absence of reinforcing, their failure is very brittle. In all loading cases, they could only resist the loads from 50% bearing loss, and sometimes were exceeded by this too. For lateral spreading the pre-1930 section fails between 3-4.5m of dragging, depending on the cladding combinations. Their low strength and brittle failure mode, combined with the fact that they often use heavy roof and wall cladding materials, means concrete perimeter houses overall are critically susceptible to serious damage as a result of liquefaction. The most common age group of concrete perimeter houses inspected for this research were pre-1930s. These results show that the weak and brittle nature of these old foundations is likely to have significantly influenced the poor performance of concrete perimeter foundations that was found in Chapter 4.

Concrete perimeter foundations from 1930-59 performed the best under the liquefaction related loading scenarios modelled, as they had the highest strength section, which was not reduced for light claddings, and as the section is symmetrical, it has the same strength for loading in both directions. For light claddings, this section can withstand all lengths of bearing loss for an 80% reduction in spring stiffness, and significant lengths of bearing loss for a 96% reduction. However it still fails between 5m and 10m of drag from lateral spreading and will suffer damage under liquefaction with

heavier claddings. Although seven houses were inspected for this research with foundations in this category, they did not appear to perform any better than the foundations of other ages. Four of the seven inspected had overall differential settlement of over 100mm, requiring foundation rebuilds. This suggests that concrete perimeter foundations performed badly whatever their age and section strength, and that this foundation system performs poorly under liquefaction loads. The higher strength of the 1930-59 section allows the perimeter beam itself to resist higher loads, but interior floor humping and overall differential settlements can still occur.

Foundations from the latest three standards periods, i.e. foundations built after 1960, were very similar in performance, being considerably better than pre-1930s foundations but not up to the standard of 1930-59 foundations. They had almost identical section strengths both for light and heavy claddings and similar strengths for reverse loading, as shown in Table 6.8. Due to their upside down T-shaped sections, they had lower strength for negative moments, where tension is on the top. This meant that negative moments were the most critical case for these foundations, as there was very little difference in the maximum moments produced in the different loading situations modelled, whether for positive or negative moments.

These findings support the general trend found in Chapter 3 that post-standard houses performed better than pre-standard houses. This is clearly due to the very low strength and ductility of pre-1930s concrete perimeter foundations.

For the three post-1960 foundations, it was shown that wall cladding does not affect the foundation performance. This is because the increased demands of heavy claddings are matched by increased section strength, and in fact in some cases light wall claddings were the more critical case, as the sections were weaker in proportion to the demands with heavy roof cladding. This agrees with the findings in Chapter 4, and also holds for pre-1930s foundations, as these are so weak that at the low end of the scale the demands do not change as significantly between light and heavy claddings. Although it was not modelled here, this trend is expected to extend to two-storey houses when compared to one-storey houses, as for the newer foundations section strength is also increased for more storeys. This is supported by the findings in Chapter 4 and in Chapter 3 that ‘light’ and ‘heavy’ buildings had little difference in damage. Wall cladding affects the performance of 1930-59 foundations, as their section strength does not change, but as their section strength is already the highest, it is more of an advantage to have light claddings than a disadvantage to have heavy claddings. As shown in Appendix A, the section capacity of these foundations is still increased for two-storey houses, again supporting the trends in Chapters 3 and 4.

The roof cladding does have an influence on foundation performance, as heavy roof claddings increase the loads on the foundation, and section strength is not increased based on this. This effect is more significant for houses with light wall claddings, as the level of load increase remains the same,

so is larger proportionally for the smaller loads with light claddings. Because of this it is recommended that heavy roof claddings in particular should not be used for new construction. Although wall cladding has been shown not to affect the performance of foundations, it is still advisable to also stay away from heavy wall claddings, as the cladding itself is more easily damaged, and inertial shaking effects can be worse, as found by Buchanan & Newcombe (2010) and Buchanan et. al. (2011).

The simple analysis of lateral spreading performed here shows the extreme loads caused by lateral spreading. There is total failure after less than 10m of drag for all concrete perimeter foundation sections even with light cladding and roofing, which reduces to only 3.5m for the post-1980 sections with heavy cladding and roofing. The significant moments caused by liquefaction add to these demands, effectively reducing the section capacities due to moment-axial interaction, meaning foundations can completely fail under little dragging force.

6.4.1 Applicability of Results

There were no significant problems encountered with the modelling procedure used. The main issues with the results stem from applicability to the real life situation, due to the simplified nature of the modelling approach. The only real issue was what section sizes to use. As discussed in Section 6.2.1, a representative depth of 600mm was decided upon, along with a width of 200mm, with reinforcement layouts based on the appropriate standards for each time period. Although there was an infinite range of choice for the dimensions, it was deemed appropriate to choose one representative size, as the purpose of this modelling is to gauge the general level of performance of the different concrete perimeter foundation sections.

There are a number of issues surrounding the applicability of the results of this modelling with any real life situations. The use of linear-elastic soil springs meant the method for creating the loading situations theorised in Section 6.2.2.2 was to degrade the spring stiffness of a certain number of springs over varying lengths. To do this the degradations of 50%, 80%, 96% and 100% were made. In reality the loss of bearing capacity at the ground surface where residential foundations are located is more complicated, with the influence from any crust layer and the degree and depth over which soil softening or liquefaction occurs causing a wide range of scenarios at the surface. The main loadings caused by significant liquefaction are in reality more related to the large displacements that structures are subjected to, due both to differential ground settlement, caused by varying soil profiles and conditions, and the differential settlement of the structure itself. These deformations cause large stresses in the structure, resulting in damage. These displacements, both of the ground and the building, were considered too complicated to model here, as they require non-linear, inelastic modelling of the soil and foundation, turning the problem into complex soil structure interaction

outside the scope of this research. As a result of this the degradations in spring stiffness were made as a simple way of replicating the hypothesised loading scenarios.

A related issue was the value chosen for the soil spring stiffness, which was based on broad assumptions of the soil type and properties. This was necessary as the modelling was aimed at representing the ground conditions right across the areas of Christchurch worst-hit by liquefaction, where there is significant variation in soil profiles and conditions. This lack of precision was combated by varying the soil stiffness over a large range, from 5000kN/m right down to 0.02kN/m. By doing this, the results from this modelling can be applied to a wide range of soil conditions and liquefaction scenarios based on more site-specific investigations for individual properties.

The other issue with the results of this modelling is that the concrete perimeter foundation beam was modelled elastically. This prevented displacements being examined, as the large majority of structural displacements occur after the yield of the concrete perimeter. It also meant that the moment demands found could not be compared against the ultimate capacities of the various concrete perimeter sections. However, as discussed in Sections 6.2.2 and 6.3.1, due to the low reinforcing steel contents of all the concrete perimeter sections modelled, the cracked stiffnesses are very high, similar to the initial elastic stiffnesses of each section. Because of this, it was deemed acceptable to compare the results of the elastic analyses to the yield capacities of the concrete perimeter sections. This allowed more direct comparison between the sections from different construction standards under a range of loading scenarios. Inelastic modelling would have required each section to be modelled individually, for each of the loading scenarios considered. This could be included in future research, to gain a better understanding of the displacements and soil loads causing failure, but was considered outside the scope of this research.

7 Summary and Recommendations

7.1 Summary of Key Findings

The 2010-2011 Canterbury earthquake sequence subjected Christchurch to high seismic demands, with peak ground accelerations between 0.25-0.45g recorded in the 22nd February 2011 Christchurch earthquake. The North and East of the city suffered severe and widespread liquefaction as a result of the high levels of ground shaking and the poor soil conditions in these areas. This caused significant damage to residential houses and in particular their foundations.

The two most common types of residential house foundations in Christchurch are concrete perimeter and slab-on-grade, both constructed to the housing standard NZS3604. Specialist ribraft slab and driven pile foundations are also used for residential buildings, particularly in areas with poor bearing capacity and for large and unusual houses.

The performance of houses and their foundations when subjected to the effects of liquefaction has been investigated through the analysis of inspection data. This included 500 general house inspections conducted by a University of Canterbury summer research team following the 4th September 2010 Darfield earthquake, 170 detailed inspections with measurements of floor levels and slopes conducted by the author following the 22nd February 2011 Christchurch earthquake and seven invasive inspections of houses during demolition. Simplified modelling was also conducted to better understand the demands produced by liquefaction-related bearing loss and lateral spreading, and how these compare to the capacity of concrete perimeter foundations. The following key findings have been derived from this research:

- 1) The damage to foundations is closely related to the ground damage they are subjected to. For the 500 houses, the distribution in overall foundation damage became worse as liquefaction severity increased, and for the 170 houses, differential settlements and floor slopes increased with liquefaction severity for all foundation types. Residential house foundations in New Zealand are in general weak and flexible, resulting in significant damage and deformations under the high loads associated with severe liquefaction effects.
- 2) Houses which experienced no liquefaction during the earthquakes performed well. The average equivalent slopes of the 19 detailed house inspections with no liquefaction were similar to the overall slope found in new concrete slabs by the DBH (2010). Average maximum floor slopes of these houses were also below the DBH (2011) criteria defining a 'damaged' foundation of 1/200, meaning no repair is required. Of the 500 houses inspected by University of Canterbury teams where no surface liquefaction was observed, 85% showed

no significant signs of superstructure damage. This supports previous research and shows that residential houses of the style built in New Zealand are resilient to the significant inertial loading that occurred in the 22nd February 2011 earthquake.

- 3) The typical foundation damage caused by liquefaction was differential settlement. This resulted in a variety of uniform and non-uniform tilting of floors, and caused cracking, break-up and separation in all foundation types. Concrete perimeter foundations commonly suffered local interior floor humping, caused by the differential settlement between the concrete perimeter and the interior wooden floor structure. Slab-based foundations showed gradual floor tilting, and in extreme cases broke up and tilted at different angles. The typical superstructure damage observed in houses was related to the differential displacement of their foundations. Non-uniform tilting in floors and foundations caused hogging, dishing and racking in the superstructures. This resulted in jammed doors and windows and cracking of interior and exterior claddings.
- 4) Lateral spreading caused significant foundation and structural damage. The data from the 500 house inspections showed that as the severity of lateral spreading increases, the proportion of foundations suffering moderate and high damage increases, while foundations experiencing low to moderate damage reduce in number. This is due to the relatively weak nature of residential house foundations in New Zealand, which cannot withstand the large forces and displacements associated with serious lateral spreading.
 - The typical damages caused by significant lateral spreading were the rupture and pulling apart of foundations laterally. Concrete perimeter foundations are often pulled out from under the superstructure, causing the superstructure to settle down inside, resulting in serious structural damage. Differential settlements are also common, with maximum differential settlements for houses suffering significant lateral spreading of 474mm, 202mm, 182mm and 250mm found for concrete perimeter, slab-on-grade, ribraft and piled foundations respectively. In all four cases, these differential settlements were accompanied by non-uniform tilting, resulting in significant structural damage. This shows that even the stronger, stiffer and more supported ribraft and piled foundations do not always have sufficient capacity to resist extreme deformation and loading scenarios.
 - Simplified modelling showed that significant loads can be caused by lateral spreading, with axial loads up to 390kN for 10m of drag along a concrete perimeter foundation supporting heavy roof and wall claddings. All concrete perimeter foundation sections failed under less than 10m of drag for any combination of roof and wall cladding. This reduced to less than 5m in some cases. When combined with the bending moments caused by liquefaction, failure will occur even sooner, due to the reduction in section capacity under moment and axial loading combinations.

- 5) The level of superstructure damage is closely related to foundation damage, and so is also related to liquefaction severity. For all houses inspected, as foundation damage increased, the distribution of superstructure damage became worse. When a foundation performs poorly and suffers significant damage, this is transferred directly to the superstructure causing damage to this as well. Conversely, if a foundation performs well, it can protect the superstructure from damage, even if the liquefaction severity is high.
- Two of the houses inspected during their demolitions suffered high differential settlements of 104mm and 180mm. However, because the foundations remained intact and suffered little damage, both of these houses tilted uniformly in a rigid-block manner and suffered only minor superstructure damage. This illustrates how foundations that perform well can reduce the level of structural damage, even under large deformations.
 - The connection between the foundation and superstructure was also identified as a key factor in limiting damage to both structural components. Two different houses inspected during demolitions lost this connection and as a result suffered severe structural damage.
- 6) Concrete perimeter foundations performed poorly. They were outperformed by all other types of foundations included in the inspections. This is due to their weak, flexible foundation structures which easily deform under the high loads and displacements that are produced by liquefaction. Their various structural components, including the concrete perimeter wall, the short interior piers and the wooden floor and sub-floor structure do not act as a cohesive unit, due to a lack of rigid connections between each component. This makes the foundation vulnerable to differential movements which result in significant structural damage.
- Concrete perimeter foundations performed worse than slab-on-grade foundations, with 25% suffering high damage when subjected to high liquefaction severity, compared to 13% for slab-on-grade.
 - The median values for the equivalent slope and overall differential settlement of concrete perimeter foundations exceeded the DBH (2011) criteria for a 'non-damaged' foundation of floor slope less than 1/200 and differential settlement less than 50mm at all levels of liquefaction, suggesting that even for minor liquefaction where ejecta did not break the surface, repairs to the foundation were likely to be required.
 - The maximum local floor slopes for concrete perimeter foundations were also very high, regardless of the level of liquefaction severity, with median values all above 1/50 floor slope. This results in their unique deformation modes of local room and overall floor humping. These high local slopes occur at the first sign of liquefaction,

but do not increase noticeably as liquefaction severity increases, demonstrating the vulnerability of concrete perimeter foundations to the effects of any liquefaction.

- The houses with concrete perimeter foundations inspected during demolition were more likely to suffer high to severe structural damage to the superstructure and foundation than slab-on-grade houses, even when the overall deformation mode was uniform tilting.
 - The modelling showed that there were no concrete perimeter sections based on New Zealand standards which could fully resist the demands of extensive liquefaction. The reverse T-shaped sections used in the newer standards are more susceptible to liquefaction, as it produces moment demands in both directions, and these section shapes have a reduced capacity for loading in the reverse direction. Even the highest strength 1930-59 concrete perimeter section will suffer significant damage under liquefaction and lateral spreading related loads, and cannot prevent high local floor slopes from occurring.
- 7) Slab-on-grade foundations performed well at low levels of liquefaction and considerably better than concrete perimeter foundations. This is due to the stiffer, more cohesive foundation system which better resists deformation under lower levels of liquefaction. However, they still suffered high damage once the liquefaction severity was moderate or worse. At these liquefaction levels, the weak and brittle slab breaks up, leading to high structural damage.
- For the 500 general inspections, only 10% of slab-on-grade foundations with no surface manifestation of liquefaction suffered moderate damage, the other 90% having low or no damage. However at moderate and high liquefaction severities, less than 15% of slab-on-grade foundations had no damage, with almost 50% suffering moderate or worse damage for high liquefaction.
 - For no and low liquefaction severity, the median values of equivalent slope and differential settlement for the detailed inspections were below the DBH (2011) criteria for foundations requiring repairs of 1/200 slope and 50mm differential settlement. Only three slab-on-grade foundations required repairs after being subjected to low liquefaction severity, with differential settlement higher than 50mm. This was significantly better than concrete perimeter foundations, and the median maximum local slopes were also much lower at these liquefaction levels, being close to or below the DBH (2010) observed local slope in new concrete floors of 1/150.
 - For moderate liquefaction, the median slab-on-grade foundation will need repair, with differential settlement higher than the 50mm threshold, and two foundations required rebuilds with differential settlement higher than 150mm. The two foundations

suffering severe liquefaction suffered very high differential settlements of 114mm and 202mm.

- 8) Ribraft and piled foundations performed the best. As for slab-on-grade, at low levels of liquefaction the median foundation for both these foundation types does not require any repairs, based on the DBH (2011) floor slope and differential settlement criteria. However, unlike slab-on-grade, for ribraft and piled foundations this also continues for moderate liquefaction severity. Ribraft foundations are stiffer and stronger, and piled foundations have more support against deformations, than slab-on-grade foundations, meaning they can resist higher levels of liquefaction without suffering serious damage. At extreme levels of land damage, large slopes and differential settlements are still seen in these foundation types, but it is unrealistic to expect them to withstand such loading scenarios.
- 9) Pre-standard concrete perimeter foundations performed worse than post-standard foundations. They are more likely to suffer moderate or high damage at all liquefaction levels. This is due to the weaker structural designs used prior to the introduction of the first housing standards, often with no reinforcing steel and a variety of hardfills used within the concrete perimeter walls which provided planes of weakness and reduced the mechanical interlocking ability of the concrete.
 - The modelling showed that a pre-1930 un-reinforced concrete perimeter section has an ultimate capacity of 12kNm, with a completely brittle failure mode, due to its lack of reinforcing. This is compared to post-1930 sections which have ultimate section capacities from 20kNm up to 50kNm, depending on the particular standard under which they were constructed, the type of cladding and the direction of loading. These newer sections are also much more ductile, as once yielded, they can take an increase in load of 30-40% before their ultimate capacities are reached.
- 10) The type of cladding (i.e. heavy or light weight) does not have a large affect on the performance of concrete perimeter foundations. This is because in the newer construction codes (post-1960), concrete perimeter foundations for houses with heavy claddings have larger section capacities. This means that the increased loads of heavy claddings do not largely affect foundation performance. In some cases light claddings may cause worse foundation performance, as the sections are reduced in strength by too much.
 - This trend is also seen when the foundation performance of one and two-storey buildings is compared, as in all housing standards, section strength is increased for two storey houses.
 - Heavy claddings are still likely to be damaged more easily themselves, as they are stiffer and more brittle in behaviour, and their larger seismic mass causes them to crack more easily.

- Heavy roof materials do affect the performance of foundations and therefore houses in general, as foundation strength is not increased based on the type of roof cladding. This has a larger affect on houses with light claddings, as the difference in load between roof cladding types is larger proportionally, due to the smaller load from the cladding.

7.2 Recommendations

Liquefaction is highly non-uniform and can result in large strains in the soil. These strains lead to significant global and differential settlements and horizontal displacements when lateral spreading occurs. These result in large permanent deformations and loads being imposed on foundations. To prevent serious structural damage from occurring in the foundation and superstructure, new residential house foundations need to be stiff enough and strong enough to resist these ground deformations. When the foundation performs well, the superstructure is more likely to also perform well, and overall the damage caused by liquefaction can be reduced. Because of this, it is recommended that weak, existing foundations be strengthened. Foundation details are important, and care should be taken to ensure that the various structural elements in a house are connected as rigidly as possible to each other, to allow the house to act as a unit.

Concrete perimeter foundations that are based on existing New Zealand standards should only be used where liquefaction is unlikely in an earthquake. As a whole, they are a very weak and flexible foundation system, making them highly susceptible to damage when subjected to liquefaction. This damage is transferred to the superstructure and can result in extensive repairs and foundation rebuilds being required even at low liquefaction levels.

Slab-on-grade foundations also based on existing New Zealand standards can perform sufficiently at low levels of liquefaction so as not to require structural repairs. However, once liquefaction is predicted to exceed low levels, stronger and stiffer foundations are needed to prevent serious damage.

The use of heavy roof claddings is to be avoided. These increase the demands on the foundation in all loading cases related to liquefaction, resulting in increased damage to foundations and the superstructure.

7.3 Future Research

There were a number of topics identified during this research where further investigation is needed, but was outside the scope of this thesis. More research is required on the importance of the connection between the foundation and superstructure. This research identified it as a factor that can have a large influence on the performance of the house structure as a whole, particularly at high levels of damage,

but how important it is, for each foundation type, needs more investigation. Does an increased degree of connection improve overall performance, particularly for concrete perimeter foundations, with their variety of structural components. This would allow an assessment of the value of retrofitting existing older houses with a higher degree of connection to improve performance under liquefaction. Further options for the retrofit strengthening of existing foundations should also be explored.

Modelling of slab foundations needs to be conducted. This research determined the vulnerability of concrete perimeter foundations to all loading situations related to liquefaction, but slab foundations, which were found to perform better, behave very differently and their particular modes of deformation and loads under which failure is expected needs to be investigated. Inelastic modelling of foundations is also needed to further explore the performance of foundations up to their ultimate capacities, and investigate the deformations resulting from various liquefaction-related loading combinations. These can then be related back to the detailed inspection data collected for this research.

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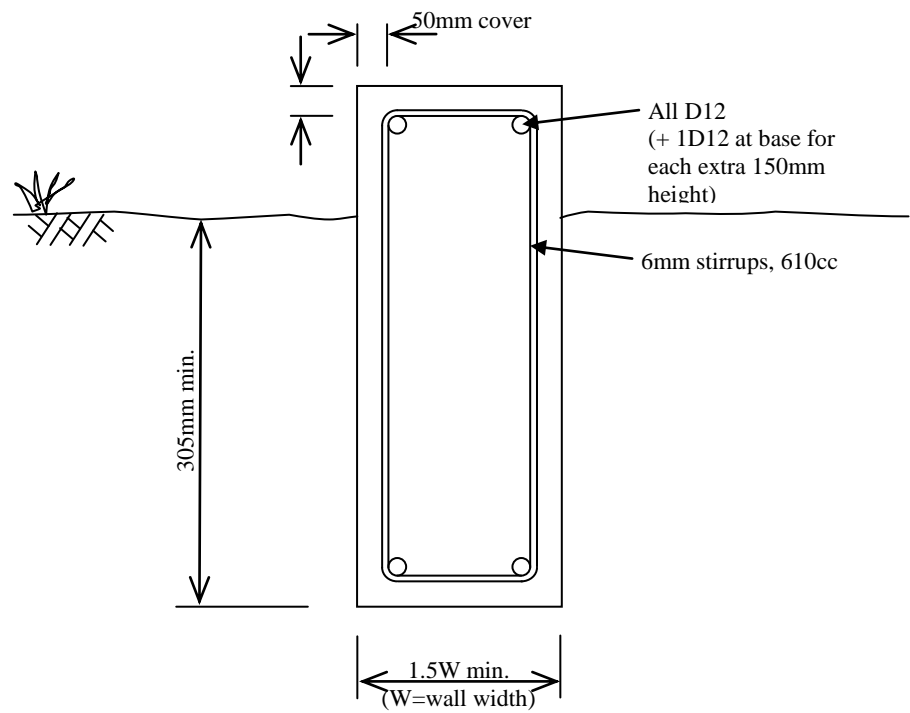
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Appendix A: Summary Foundation Drawings

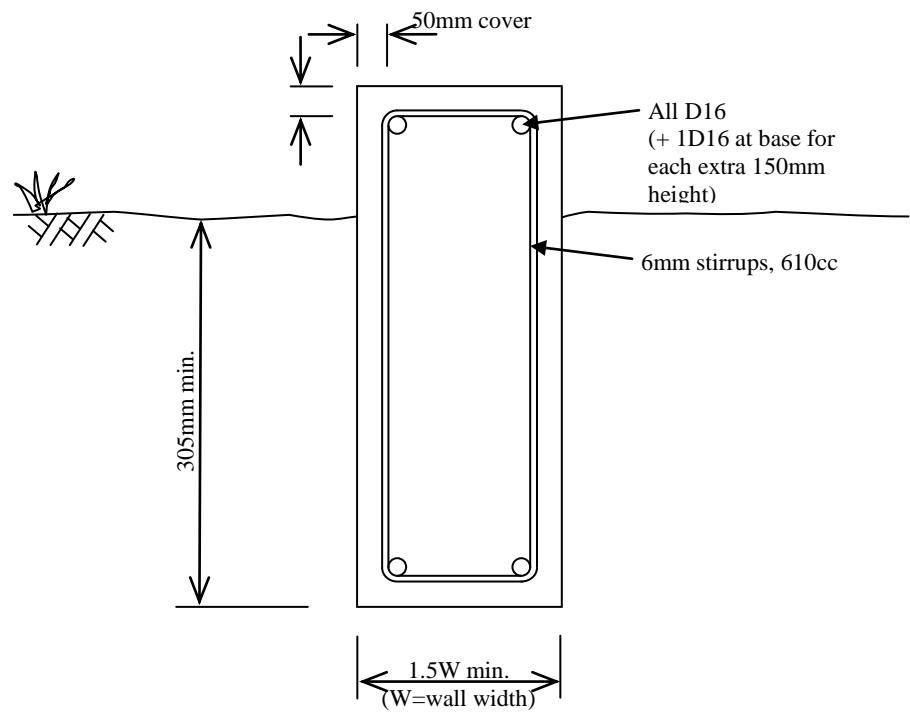
Perimeter

1930-59

1-storey:

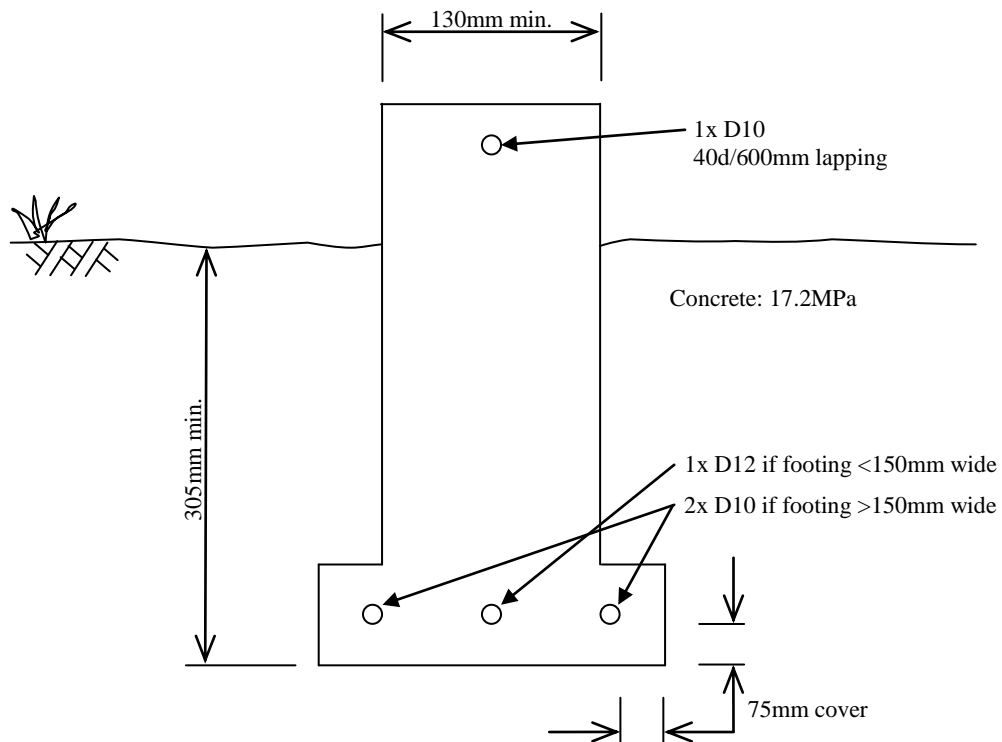


2-storey:

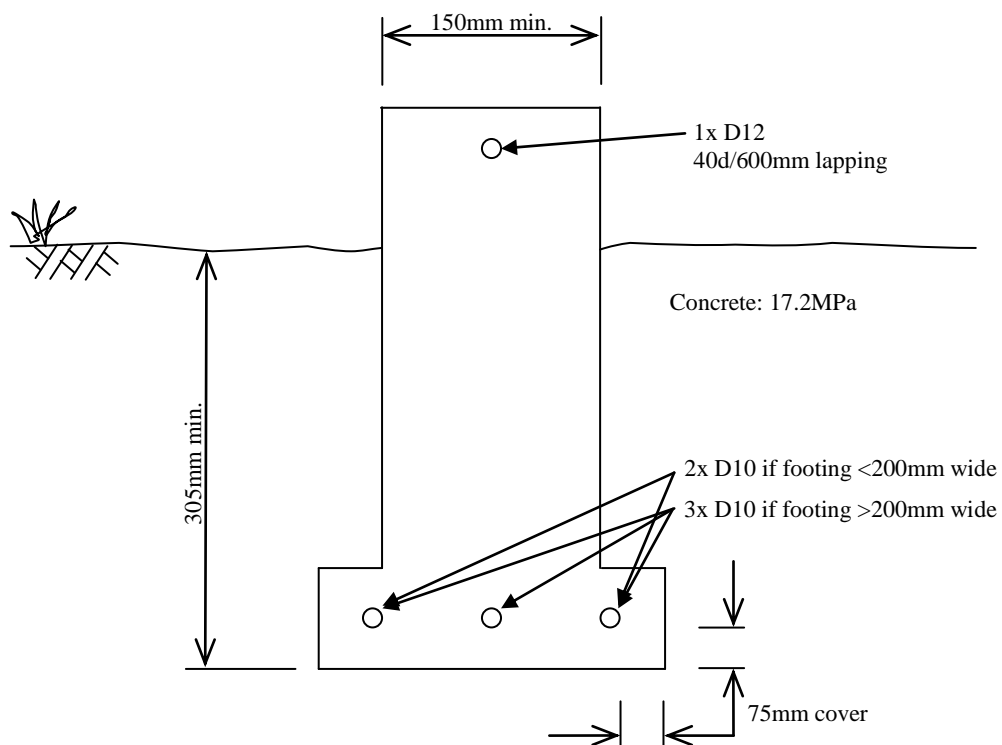


1960-79

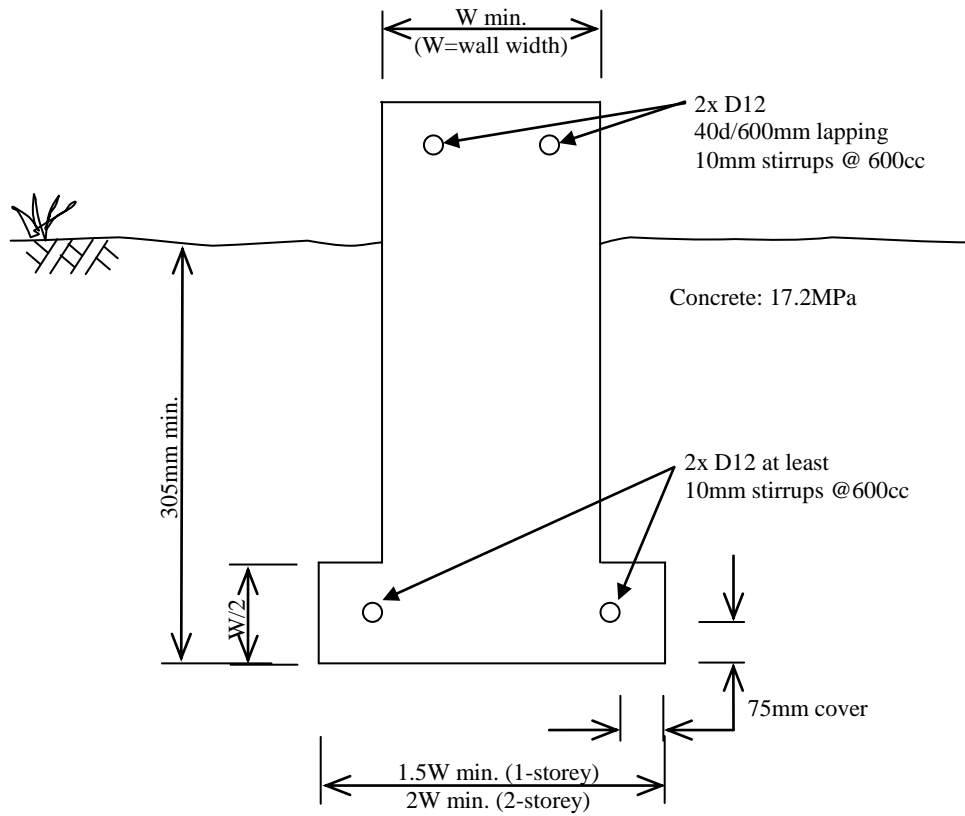
1-storey:



2-storey:

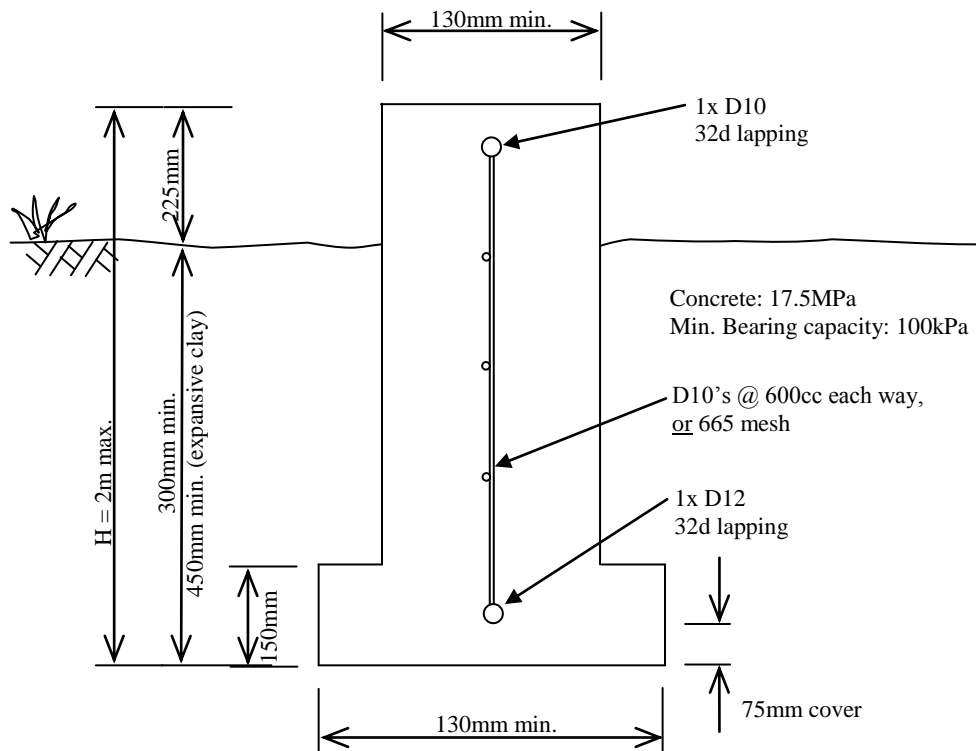


Brick:

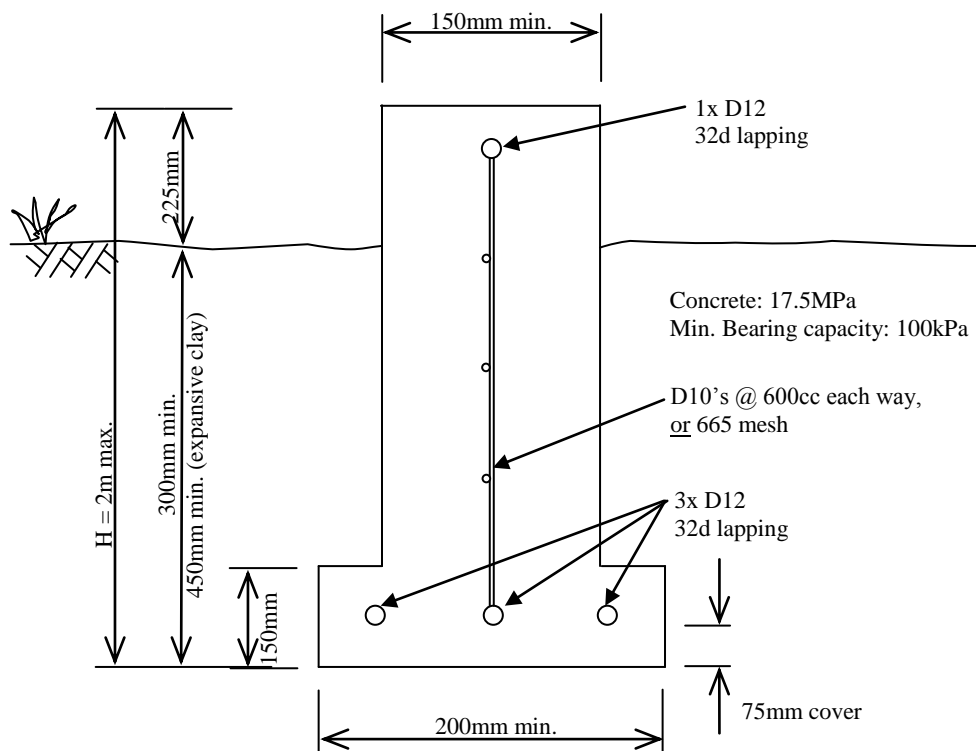


1980-99

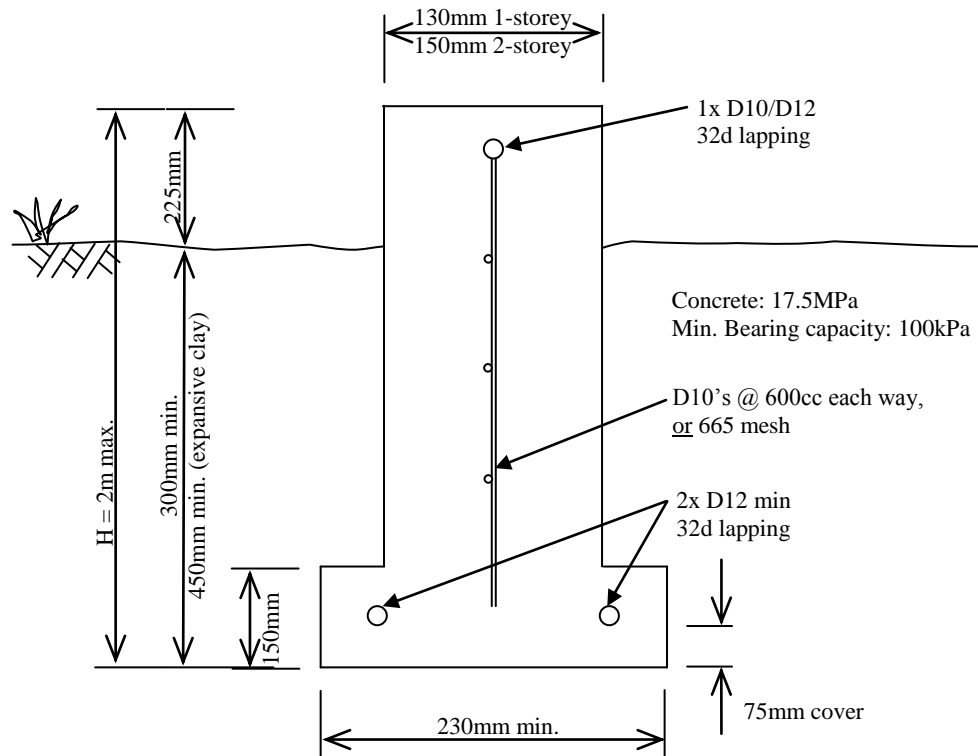
1-storey:



2-storey:

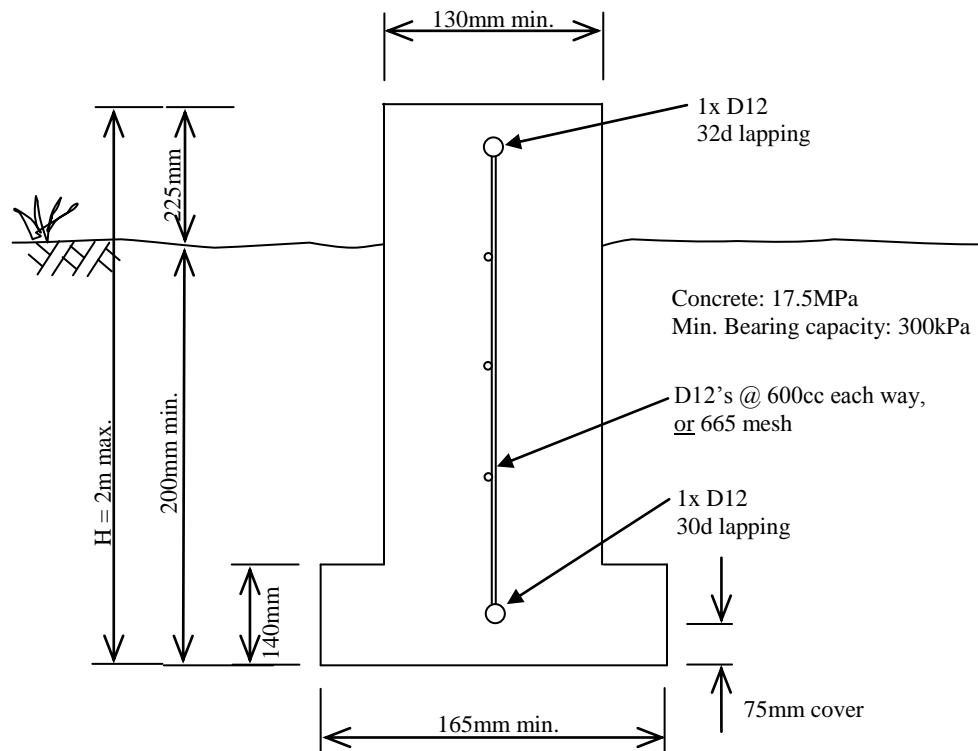


Brick:

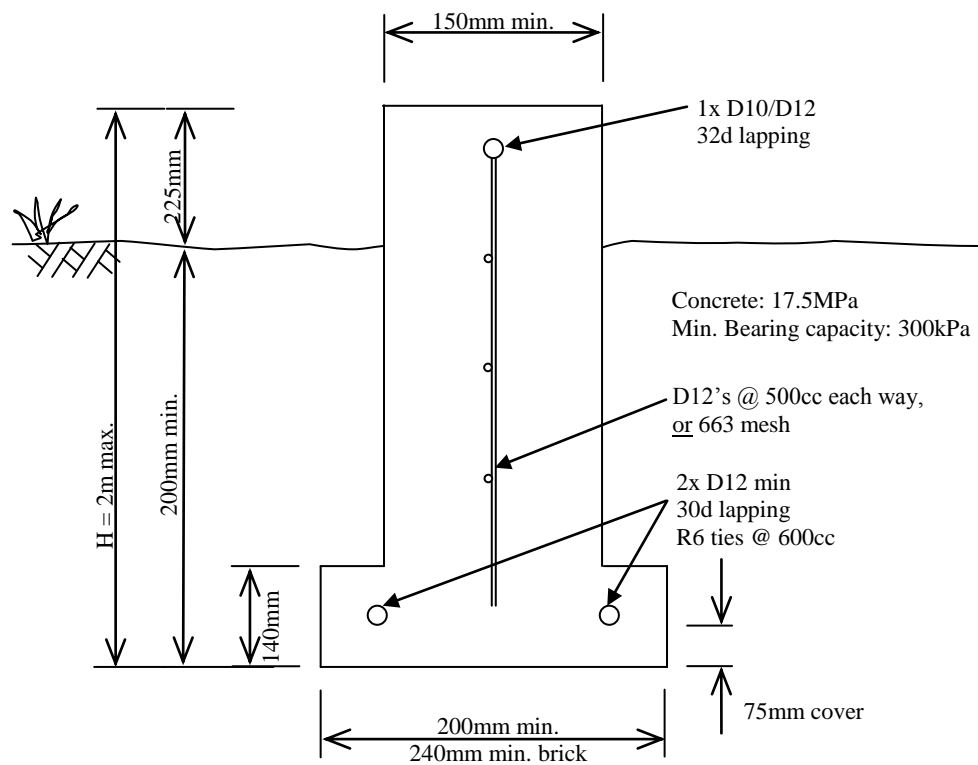


Post 2000

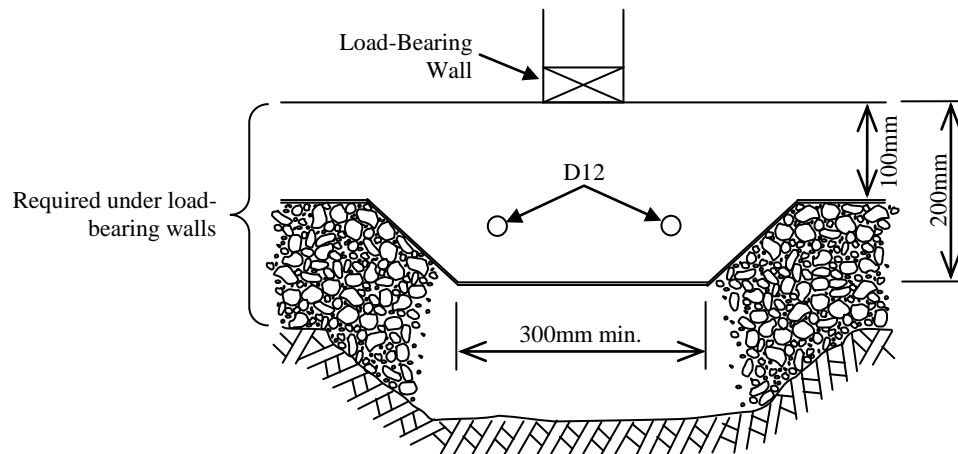
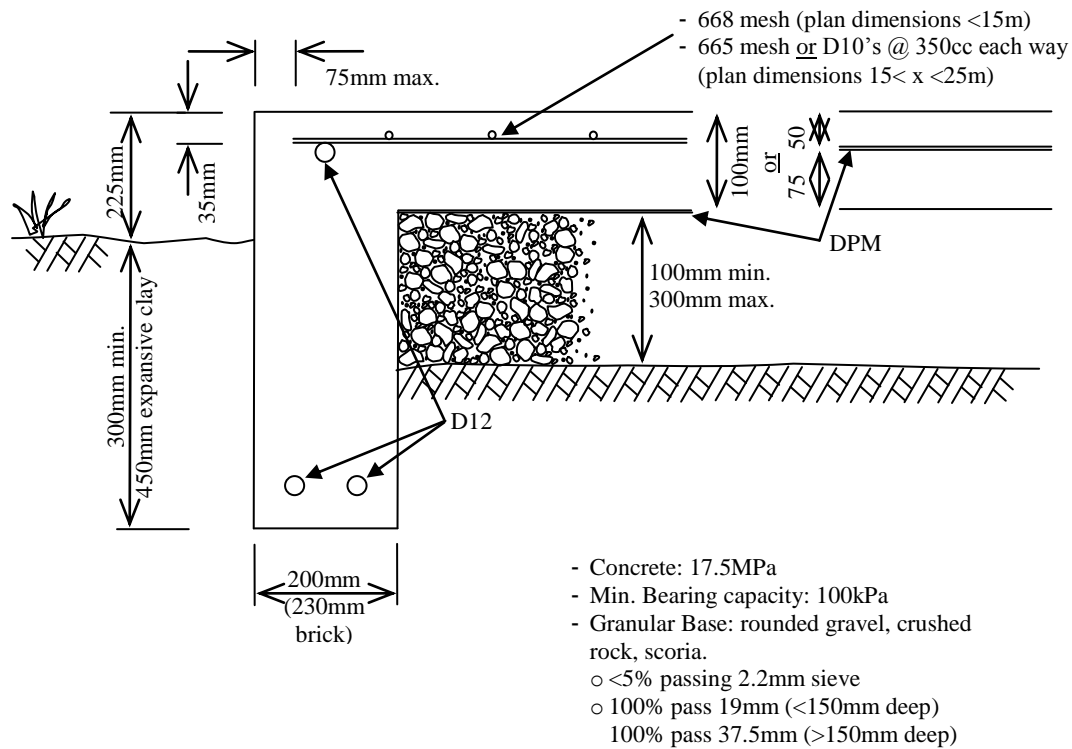
1-storey:



2-storey/brick:

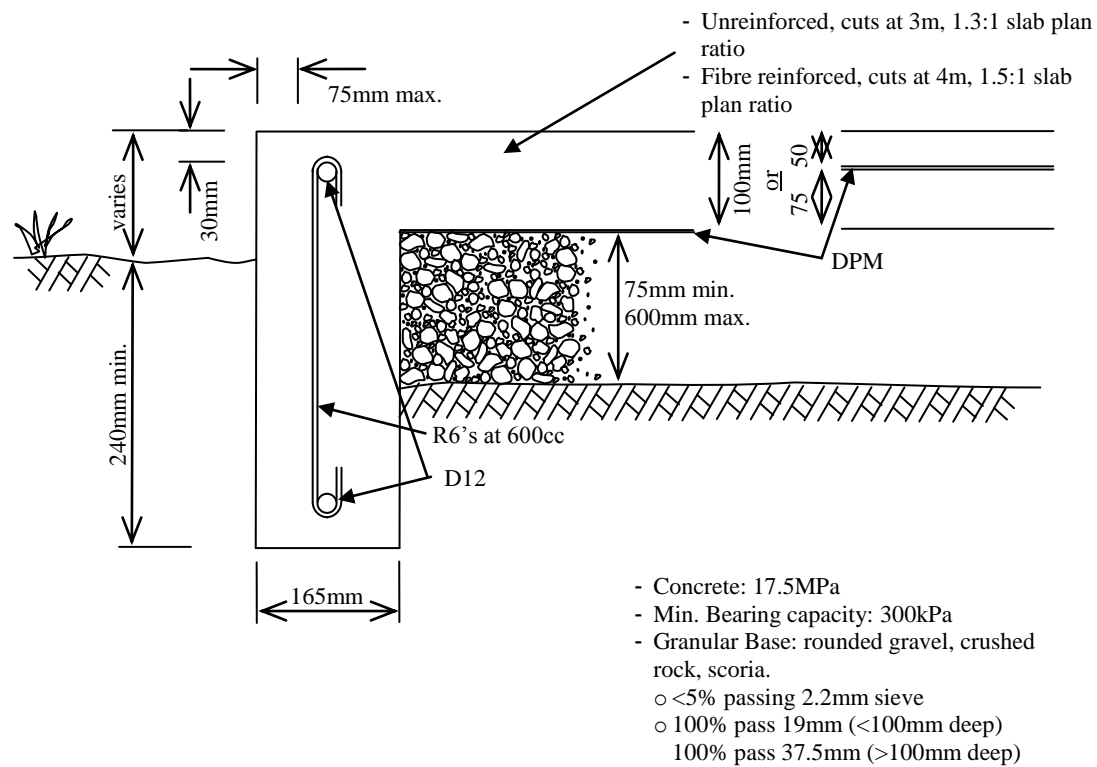


2-storey:

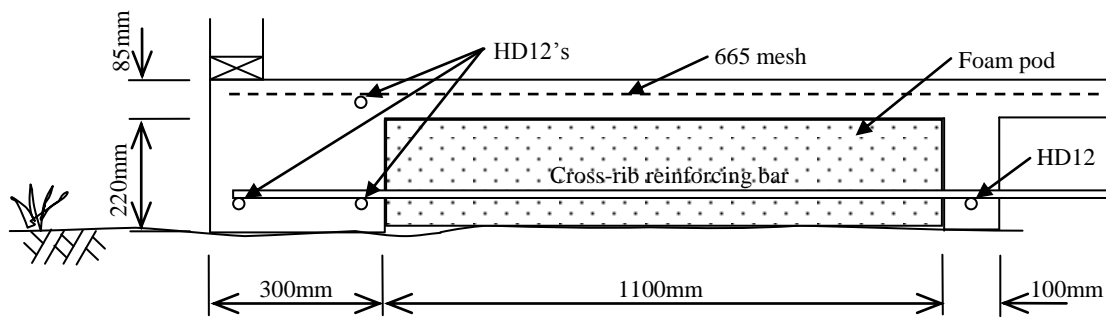


Post-2000

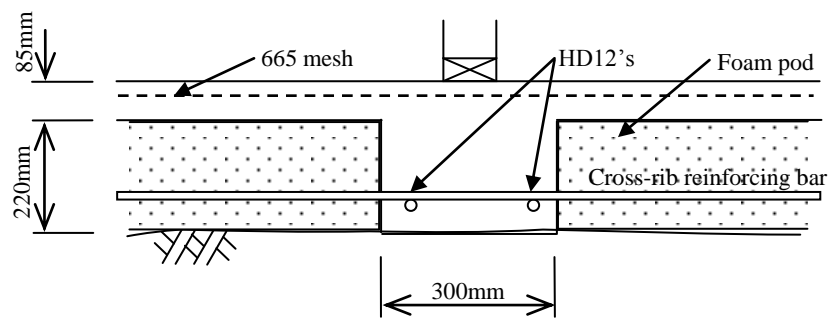
1-storey:



RibRaft



- Concrete: 20MPa
- Min. Bearing capacity: 300kPa



Appendix B: Inspection Request Form (September-February 2010/2011)

Dear owner/resident,

This letter is to ask your permission to inspect the outside of your property for research purposes, may you not be home. Please Tick the yes box if you are happy for us to do so, and leave this letter in the mailbox.

We are interested in gathering information about exterior damage to foundations, walls, and the surroundings, but we will not be entering your home.

Thank you for your cooperation.

Geotechnical research team

University of Canterbury

☐

YES

☐

NO

Appendix C: House Inspection Form (September-November 2010)

Uni of Canterbury Building Settlement Survey (7)

Area (circle):	Avonside / Darlington	Kaiapoi	Bexley / New Brighton
----------------	-------------------------------------	---------	--------------------------

By: BB, AB, DB Date: 26/9.

Address: <u>B</u>	<u>368 Avonside Dr - singlestorey, timber frame brick facade</u>			
Liquefaction Severity	<u>None</u>	Low	Moderate	High
Remarks	<u>Minor settlements according to owner</u>			
Differential Settlement	Line 1	Line 2	Line 3	Line 4
Vertical Drop				
Horizontal Length				
Tilt Angle	<u>Zero.</u>			
Total Settlement Estimate	High		Low	
Basis of estimate / Remarks				
Damage Survey	Main Structure	<u>Low</u>	Moderate	High
	External Partitions	<u>Low</u>	Moderate	High
	Foundation/Floors	<u>Low</u>	Moderate	High
	Surrounding	<u>Low</u>	Moderate	High
Other Remarks:	<u>Photos: 28 (DB), 87-88 (AB) Separation of old-new sections of house.</u>			

Appendix D: House Inspection Form (December-February 2010/2011)

University of Canterbury Building Damage Assessment Survey

Detailed Structural Damage Explanation:

- Foundation:
 - (a) settlement of piles; (b) broken piles; (c) house skewed off foundations; (d) cracking in perimeter concrete foundation; (e) cracking in slab.
- External Partitions:
 - (a) diagonal cracking; (b) vertical cracking; (c) wall tearing; (d) hairline cracking; (e) gaps opening between window/door frames and wall.
- Internal Partitions:
 - (a) diagonal cracking close to window/door frames; (b) horizontal cracking at ceiling/wall interface; (c) tearing due to lateral spreading
- Framing:
 - (a) nails pulled out between elements of frame; (b) cracking/tearing of frame elements; (c) flexural failure of frame elements
- Roof:
 - (a) buckling of spouting/gutters; (b) buckling of ridges on roof; (c) tearing between rows of tiles/sheets of iron.

Schematic Sketch of the Building:

University of Canterbury Building Damage Assessment Survey

Examination Place: 10 Robson Ave

Date: _____

Examiners: Green ☒ Yellow ☐ Red ☐

Damage Sticker: Green ☒ Yellow ☐ Red ☐

Structural Description

Foundation

- Concrete slab with perimeter footing ☐
 - reinforcing steel in slab: YON ☐ Unknown ☐
- Timber floor on short piles with perimeter concrete foundation ☒
- Timber floor on short piles with no perimeter concrete foundation ☐
- Concrete slab reinforced with driven long piles ☐

Cladding

Framing

Roof

Light weight ☒ Heavy weight ☐

Timber ☒ Concrete ☐ Steel ☐

Light weight ☐ Heavy weight ☒

Ground Damage

Liquefaction

Lateral Spreading

Remarks:

Damage Level

None ☐ Low ☒ Moderate ☐ High ☐

None ☐ Low ☐ Moderate ☒ High ☐

40mm

Building Settlement

Side 1

Side 2

Side 3

Side 4

Vertical Drop

Horizontal Length

Tilt Angle

Total Settlement (cm)

None ☐ Low ☒ Moderate ☐ High ☐

Remarks:

Structural Damage

Foundation

External Partitions

Internal Partitions

Main Structure

Surroundings

Remarks:

Damage Level

None ☒ Low ☐ Moderate ☐ High ☐

None ☒ Low ☐ Moderate ☐ High ☐

None ☒ Low ☐ Moderate ☐ High ☐

None ☒ Low ☐ Moderate ☐ High ☐

None ☐ Low ☒ Moderate ☐ High ☐

Structural Damage Type

Stretching

Hogging

Dishing

Racking/Twisting

Tilting

Diff. Settlement

Diff. Displacement

Damage Level

Low ☐ Moderate ☐ High ☐

Low ☐ Moderate ☐ High ☐

Low ☐ Moderate ☐ High ☐

Low ☐ Moderate ☐ High ☐

Low ☐ Moderate ☐ High ☐

Low ☐ Moderate ☐ High ☐

Low ☐ Moderate ☐ High ☐

Appendix E: 4th September 2010 Analysis Results

E.1: Liquefaction Damage vs. Foundation Damage

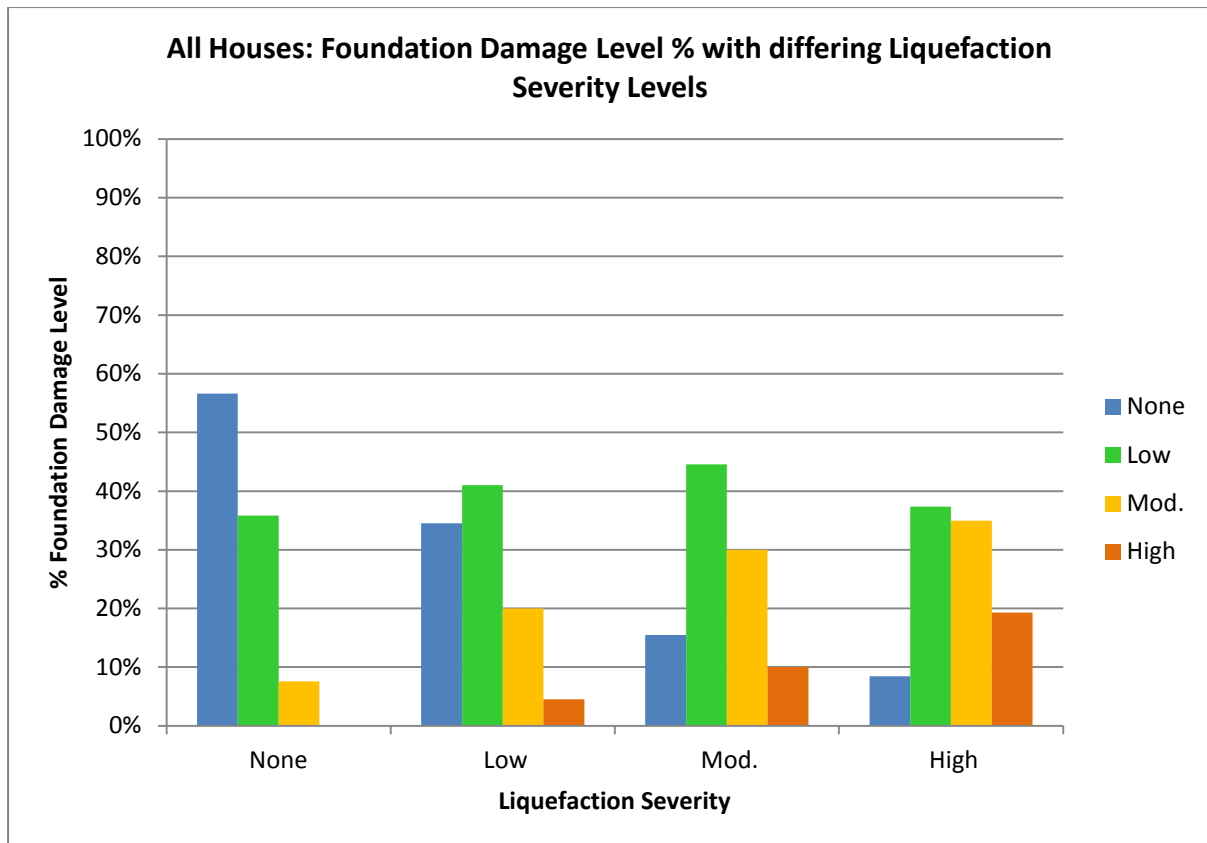


Figure A. 1. Distribution of foundation damage level in percentage for differing levels of liquefaction severity for all houses.

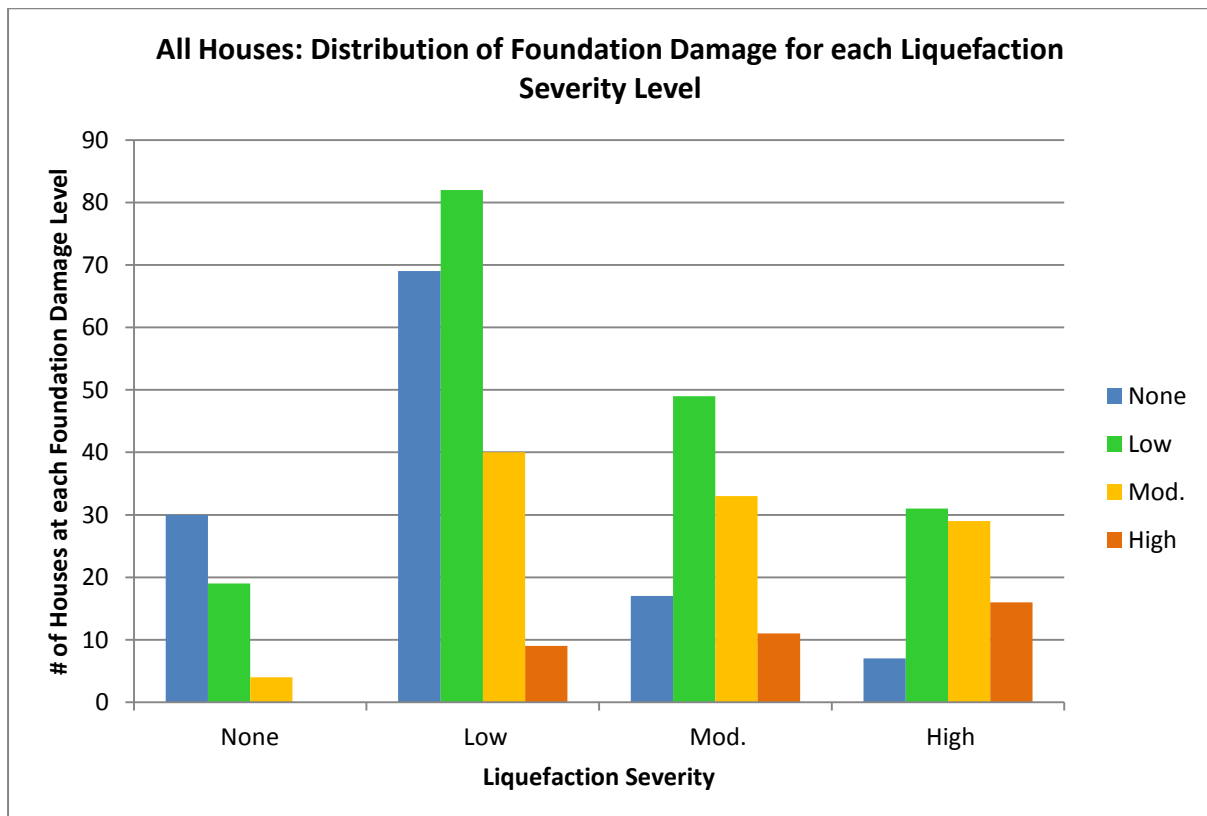


Figure A. 2. Distribution of foundation damage level for differing levels of liquefaction severity for all houses.

E.1.1: By Foundation Type

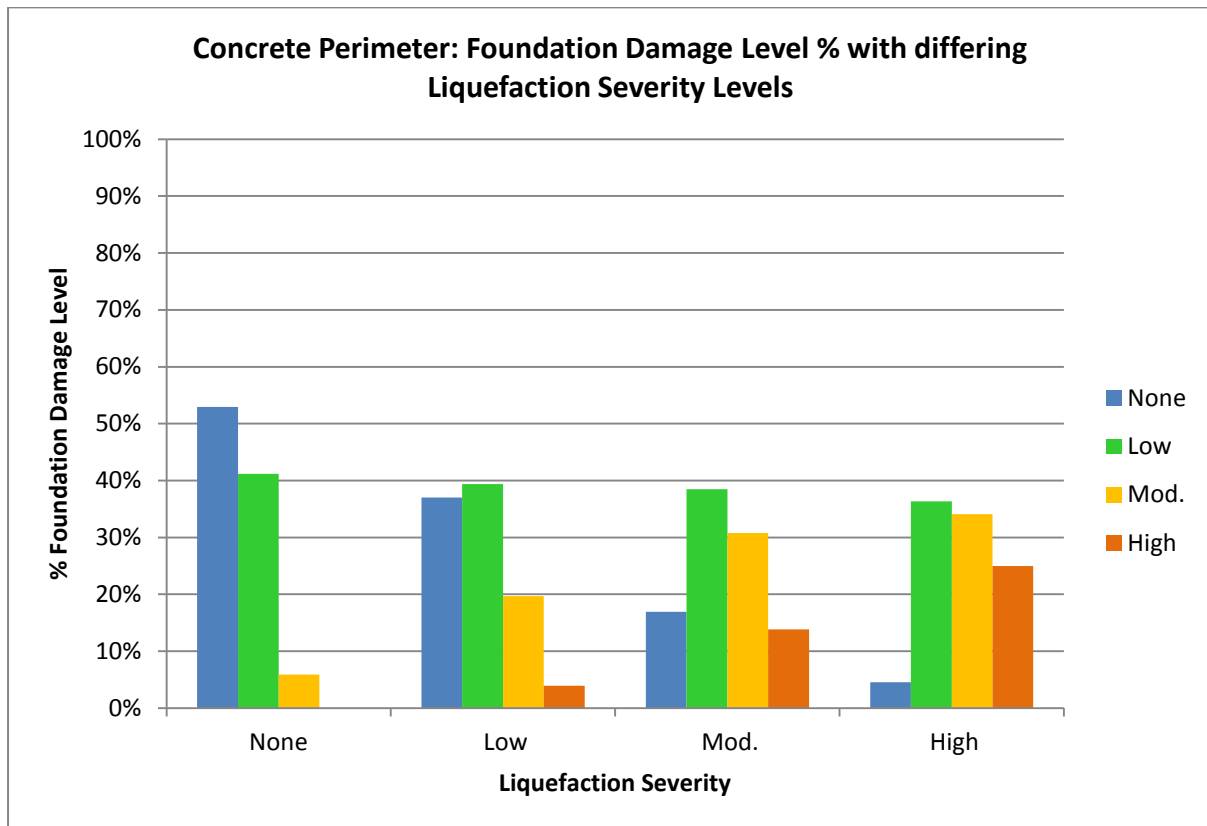


Figure A. 3. Distribution of foundation damage level in percentage for differing levels of liquefaction severity for concrete perimeter foundations.

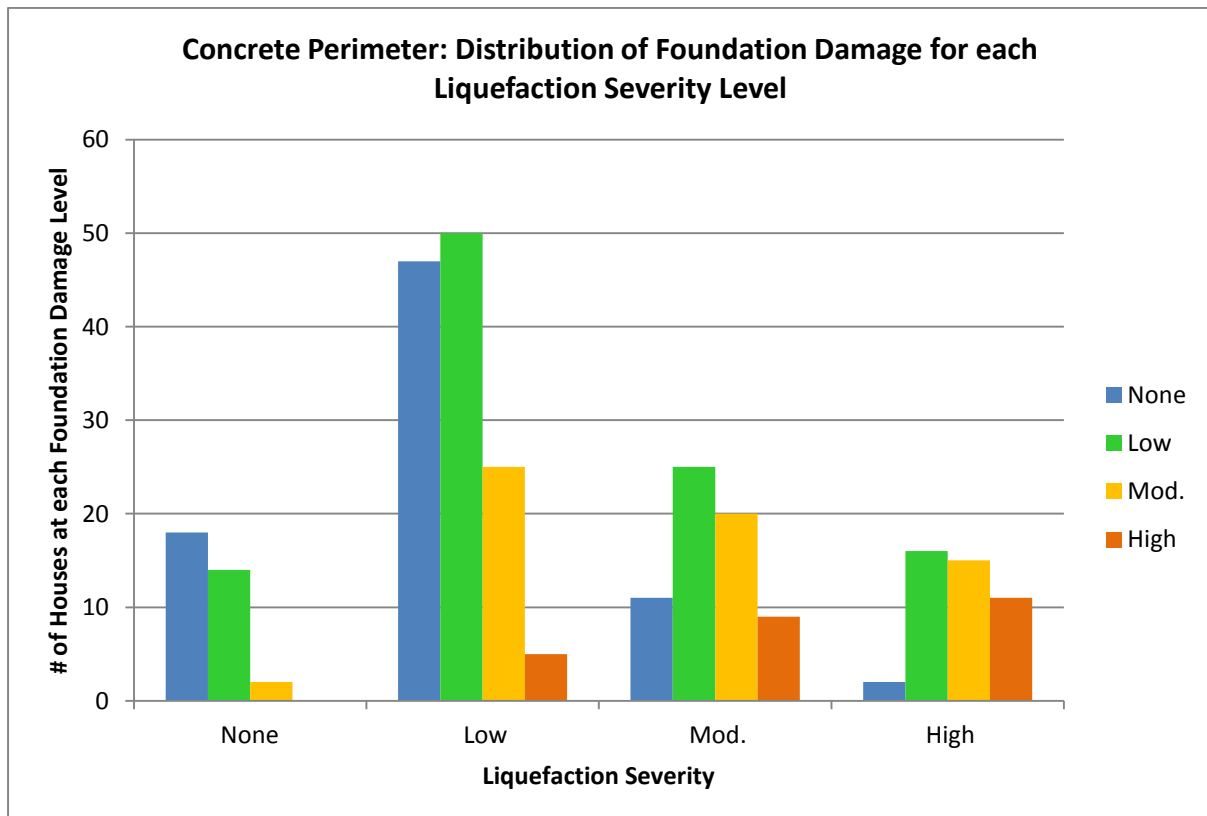


Figure A. 4. Distribution of foundation damage level for differing levels of liquefaction severity for concrete perimeter foundations.

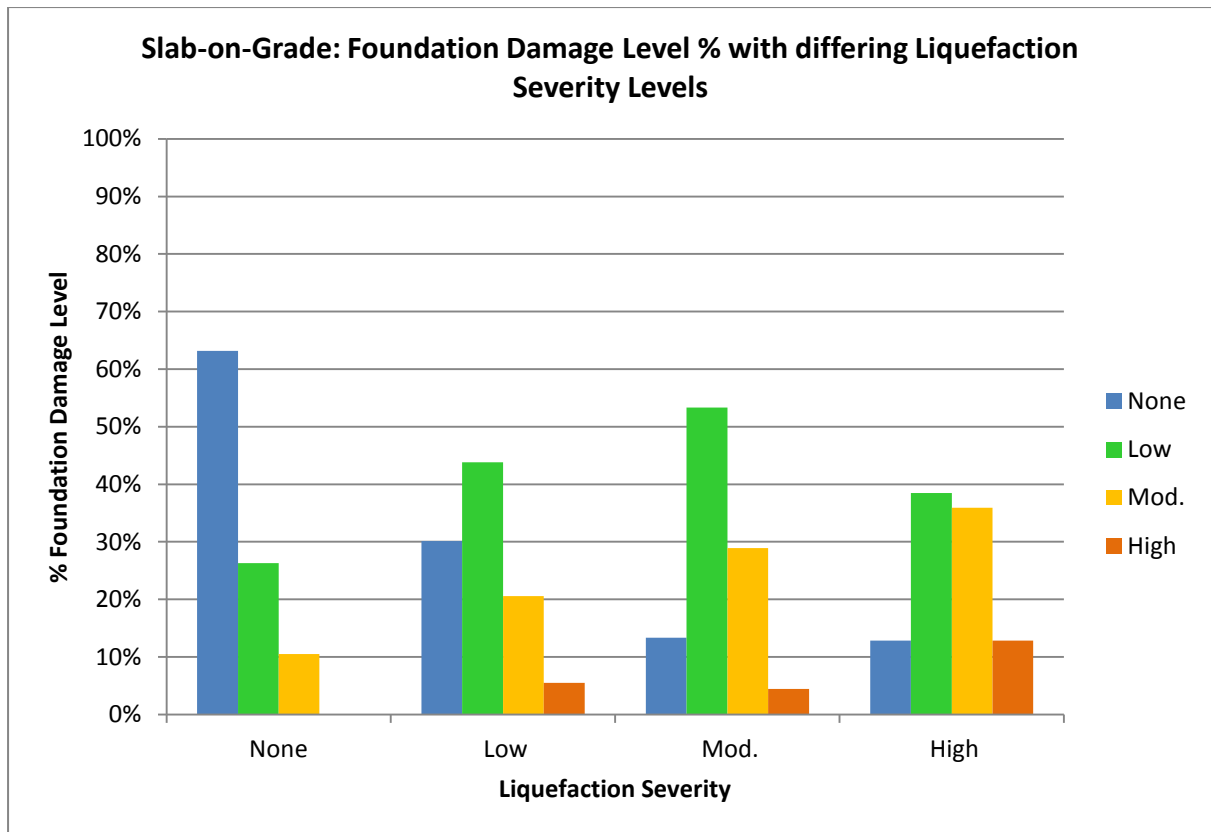


Figure A. 5. Distribution of foundation damage level in percentage for differing levels of liquefaction severity for slab-on-grade foundations.

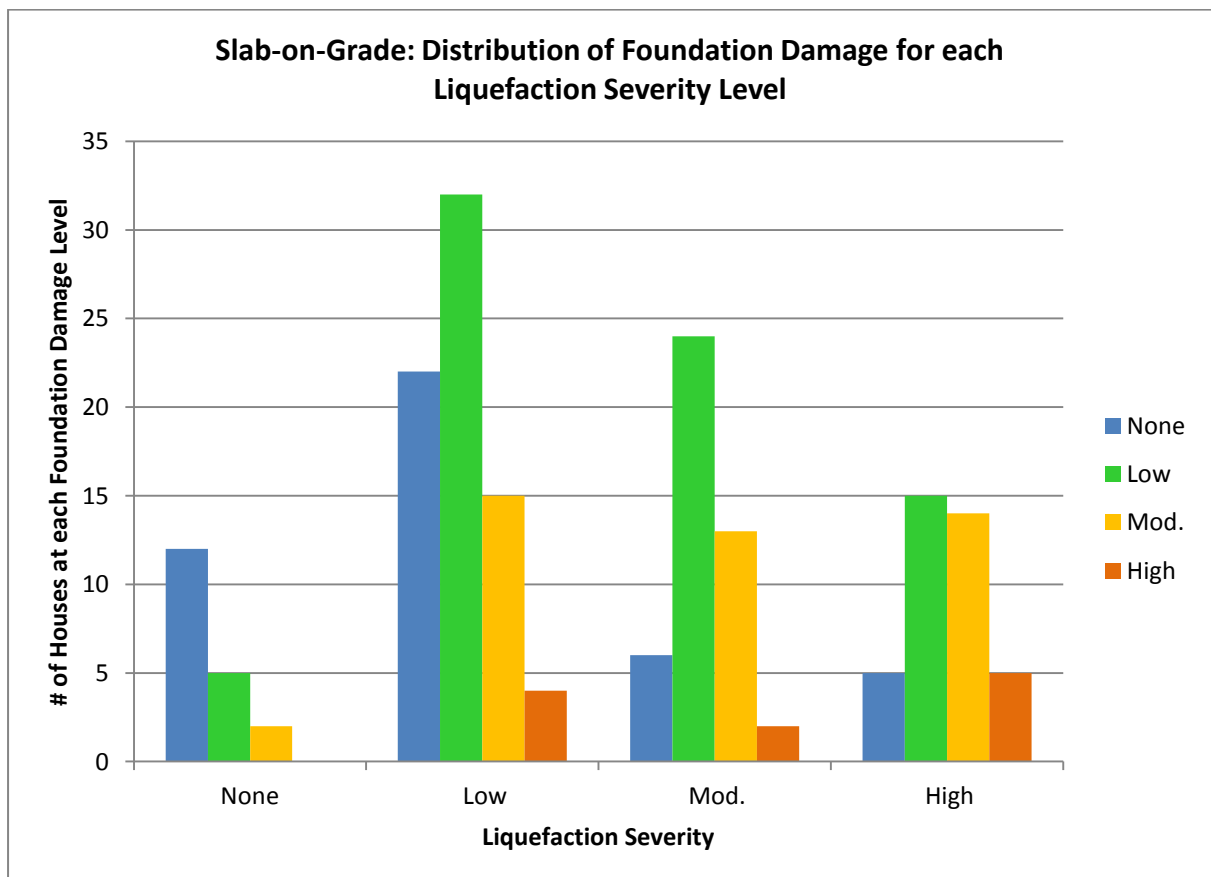


Figure A. 6. Distribution of foundation damage level for differing levels of liquefaction severity for slab-on-grade foundations.

E.1.2: By Age

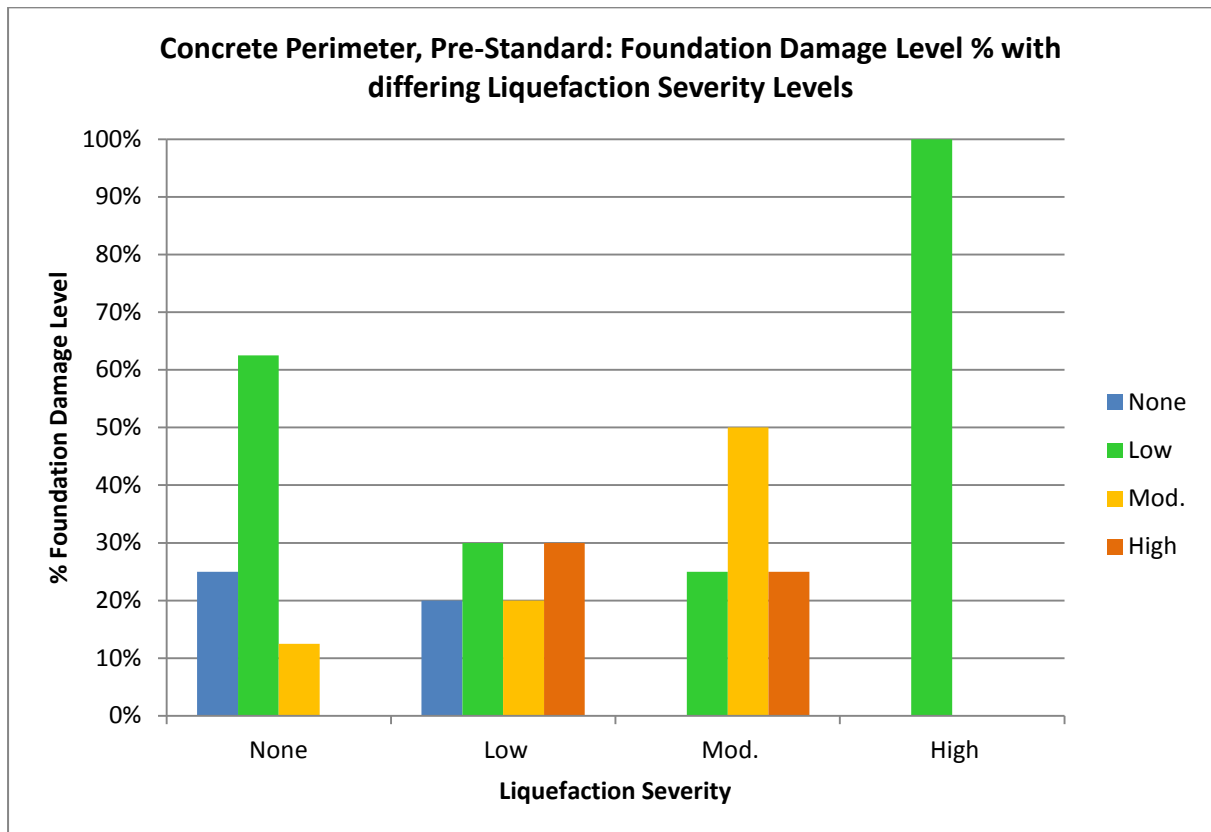


Figure A. 7. Distribution of foundation damage level in percentage for differing levels of liquefaction severity for pre-standard, concrete-perimeter foundations.

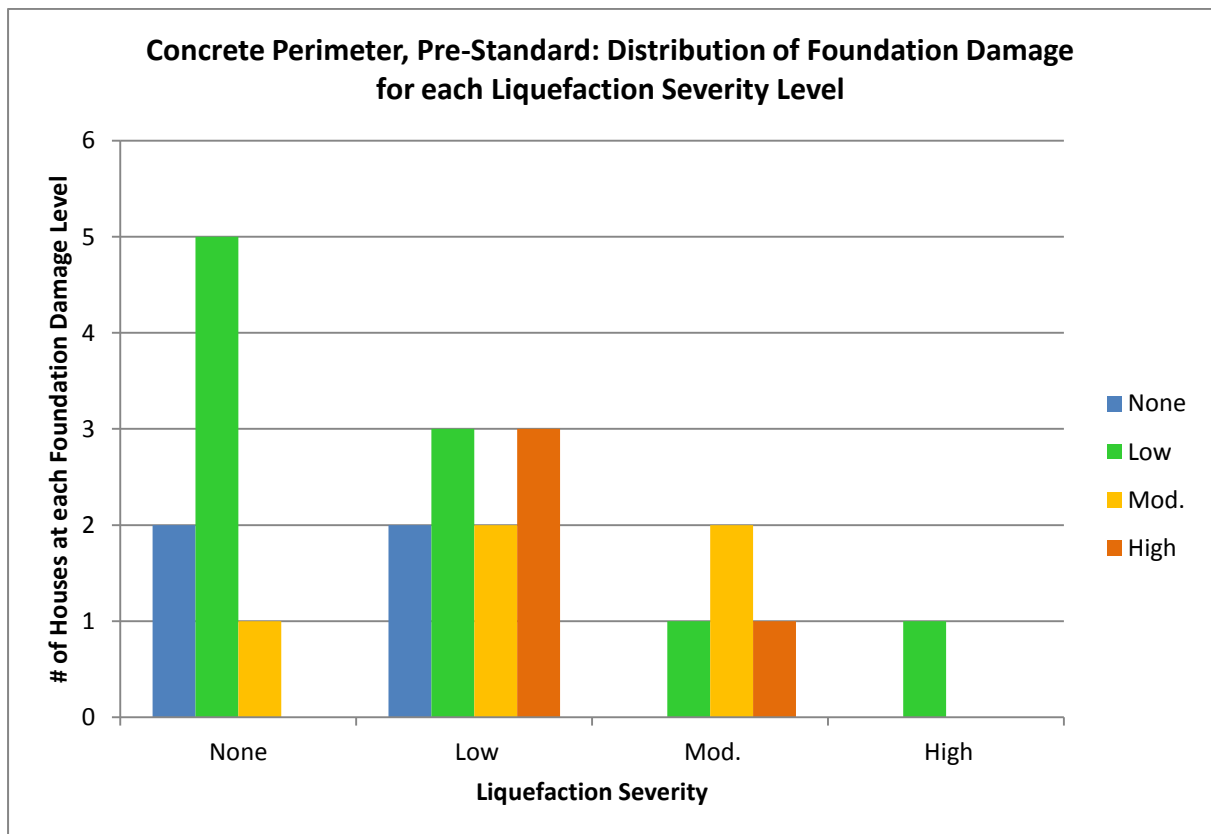


Figure A. 8. Distribution of foundation damage level for differing levels of liquefaction severity for pre-standard, concrete-perimeter foundations.

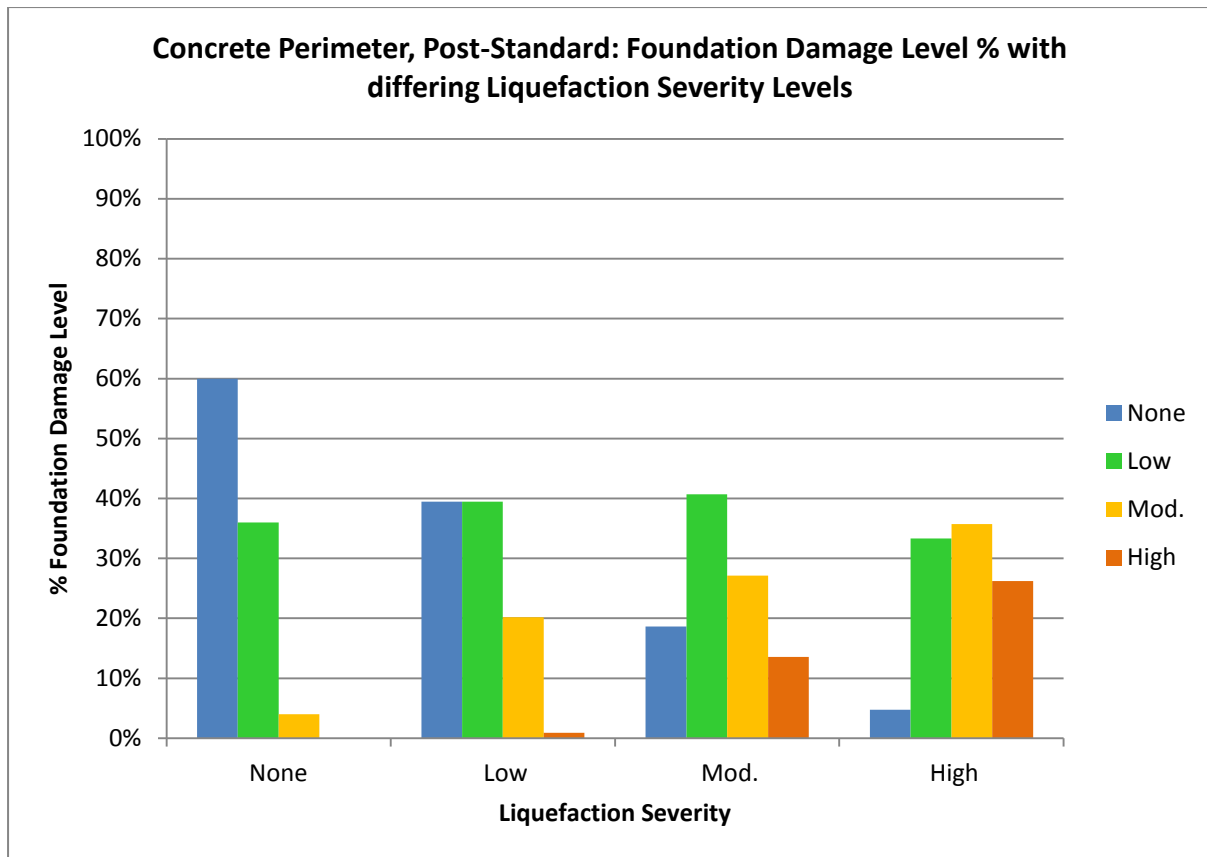


Figure A. 9. Distribution of foundation damage level in percentage for differing levels of liquefaction severity for post-standard, concrete perimeter foundations.

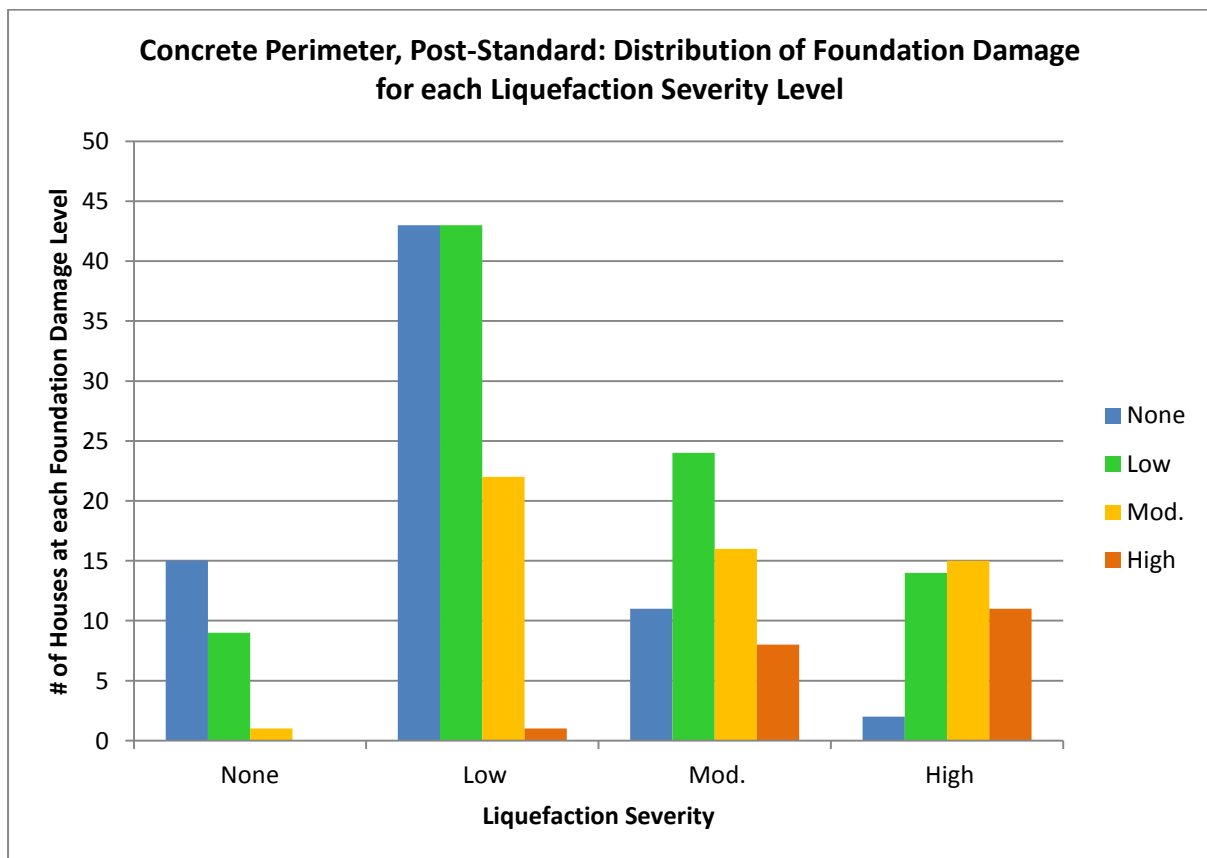


Figure A. 10. Distribution of foundation damage level for differing levels of liquefaction severity for post-standard, concrete perimeter foundations.

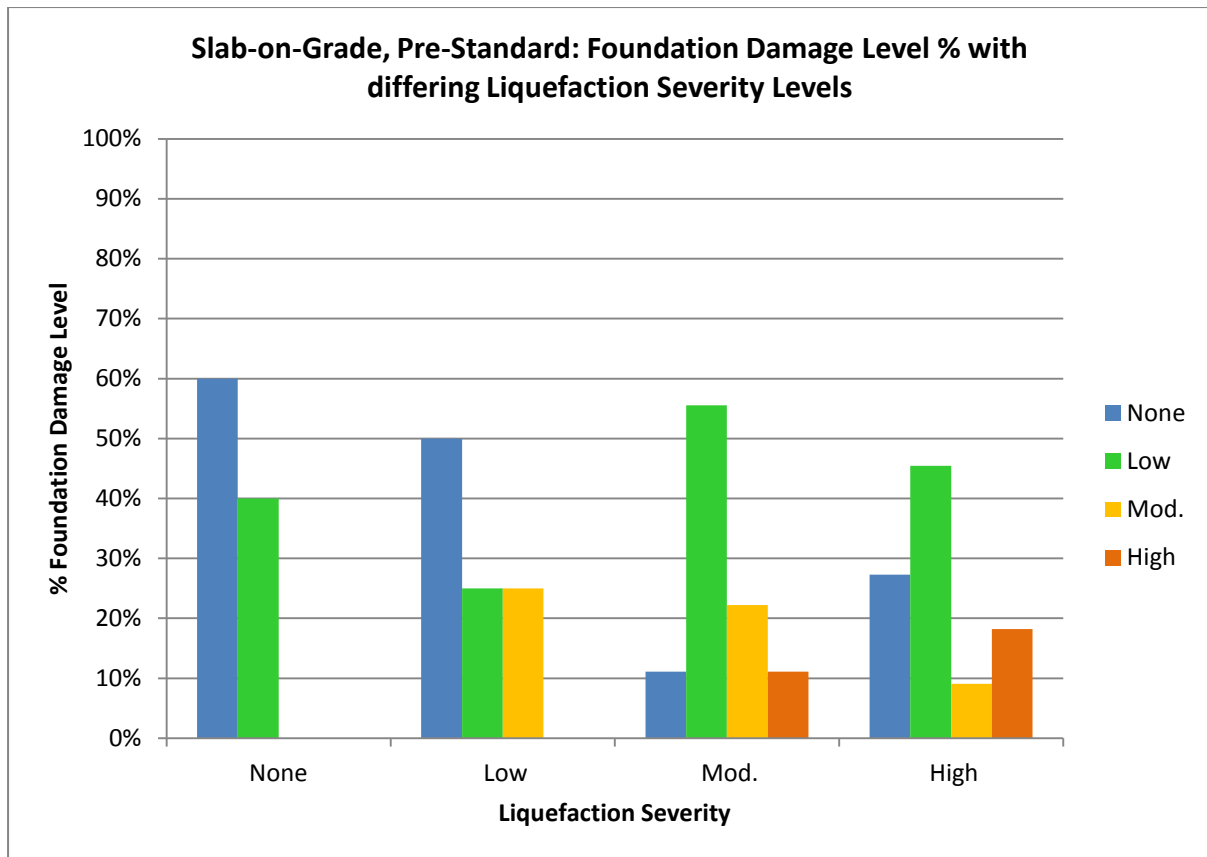


Figure A. 11. Distribution of foundation damage level in percentage for differing levels of liquefaction severity for pre-standard, slab-on-grade foundations.

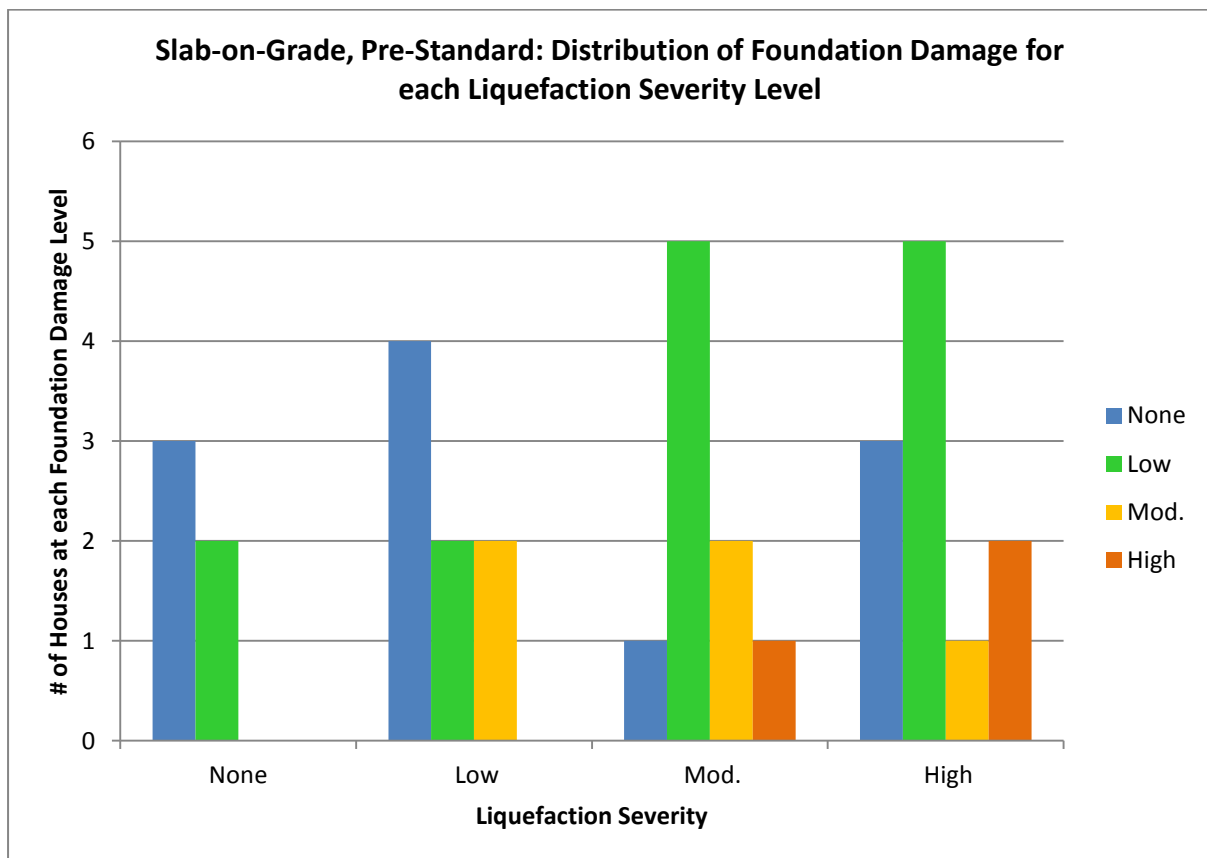


Figure A. 12. Distribution of foundation damage level for differing levels of liquefaction severity for pre-standard, slab-on-grade foundations.

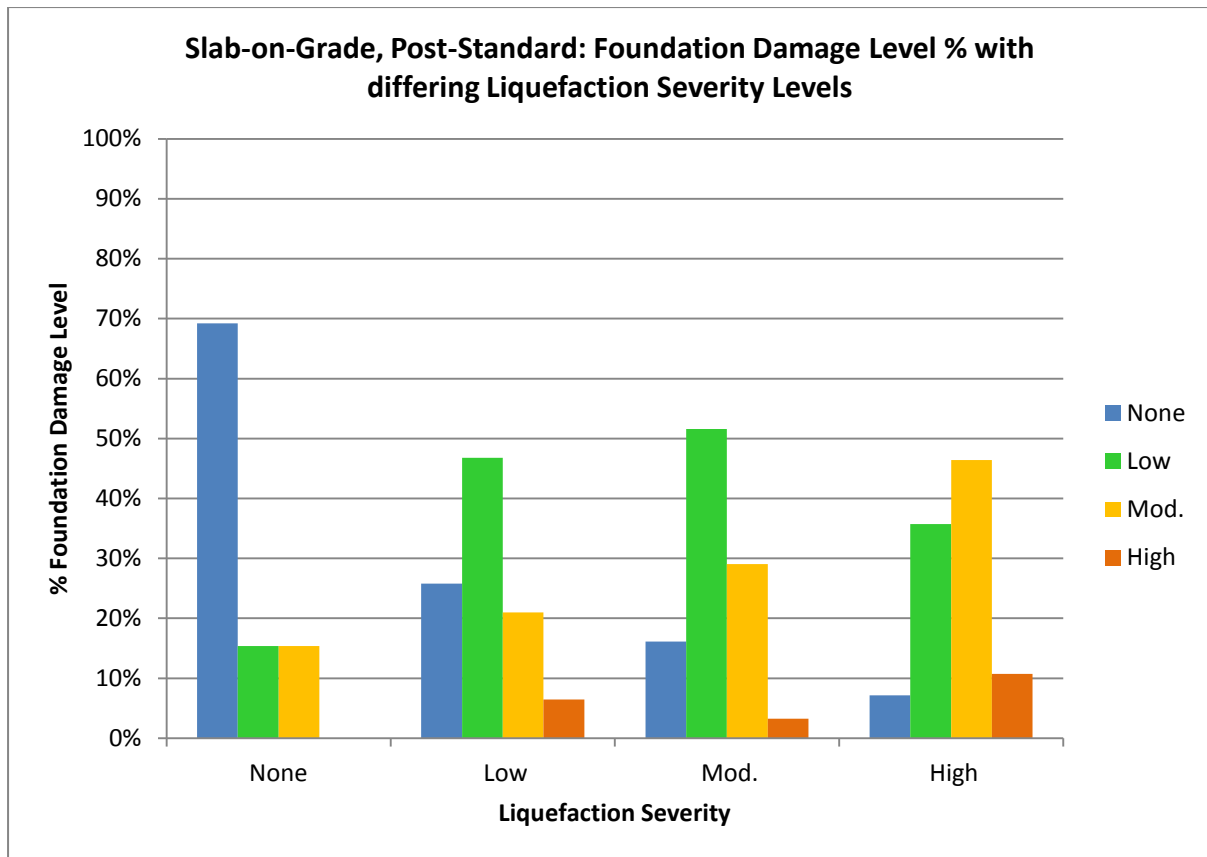


Figure A. 13. Distribution of foundation damage level in percentage for differing levels of liquefaction severity for post-standard, slab-on-grade foundations.

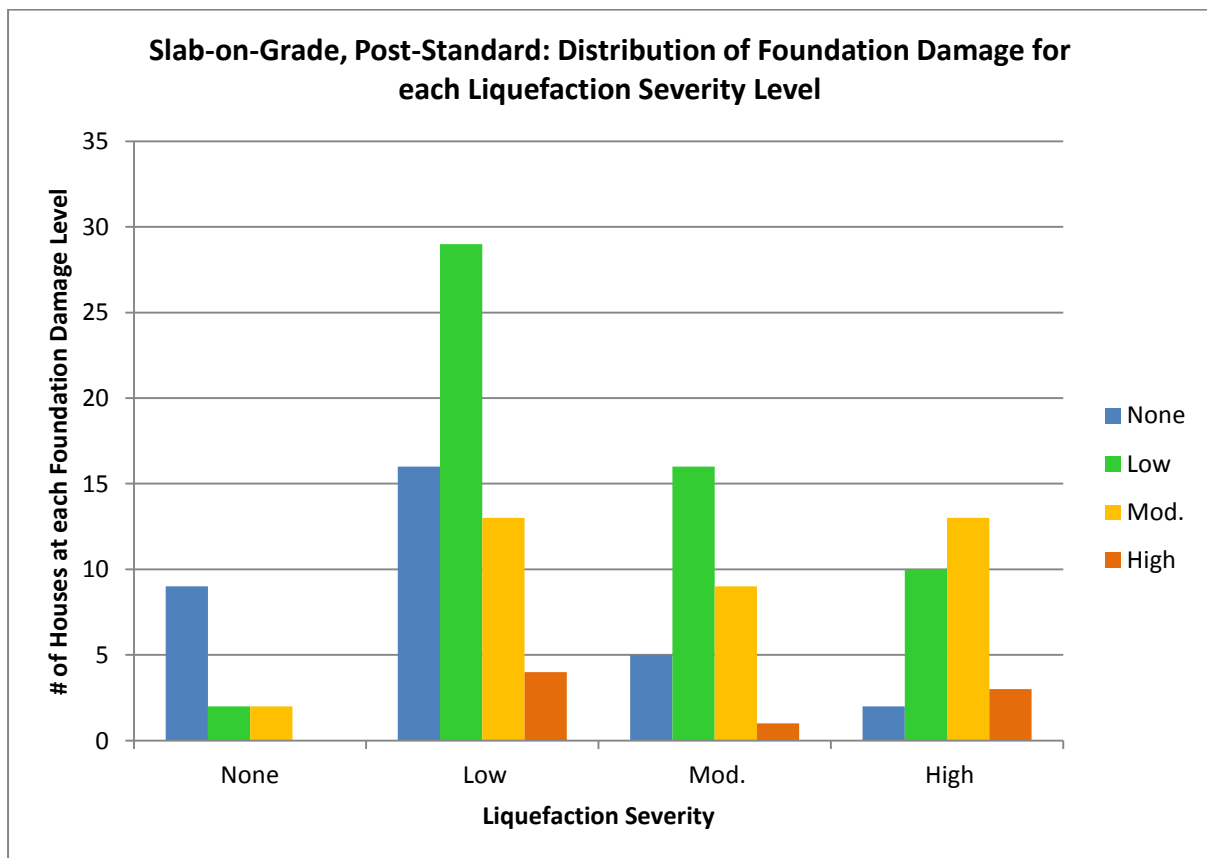


Figure A. 14. Distribution of foundation damage level for differing levels of liquefaction severity for post-standard, slab-on-grade foundations.

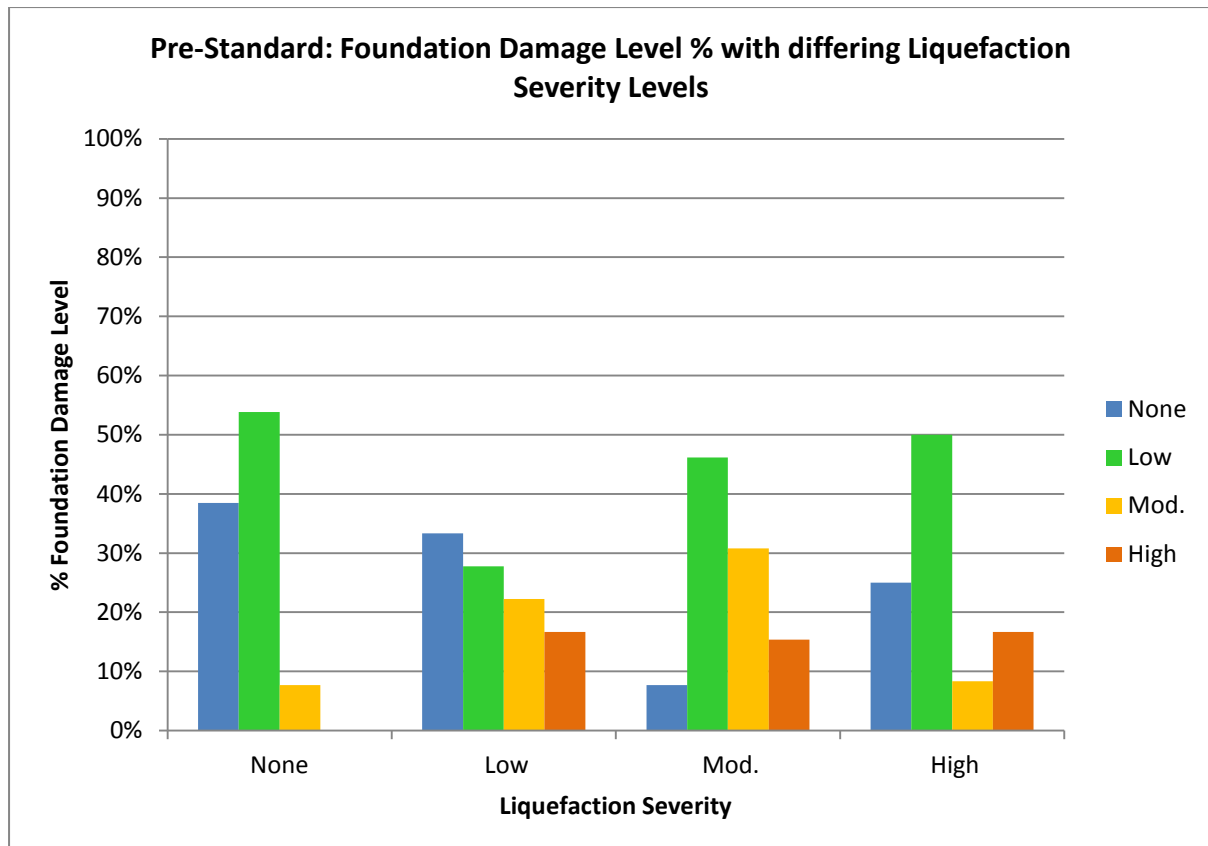


Figure A. 15. Distribution of foundation damage level in percentage for differing levels of liquefaction severity for all pre-standard houses.

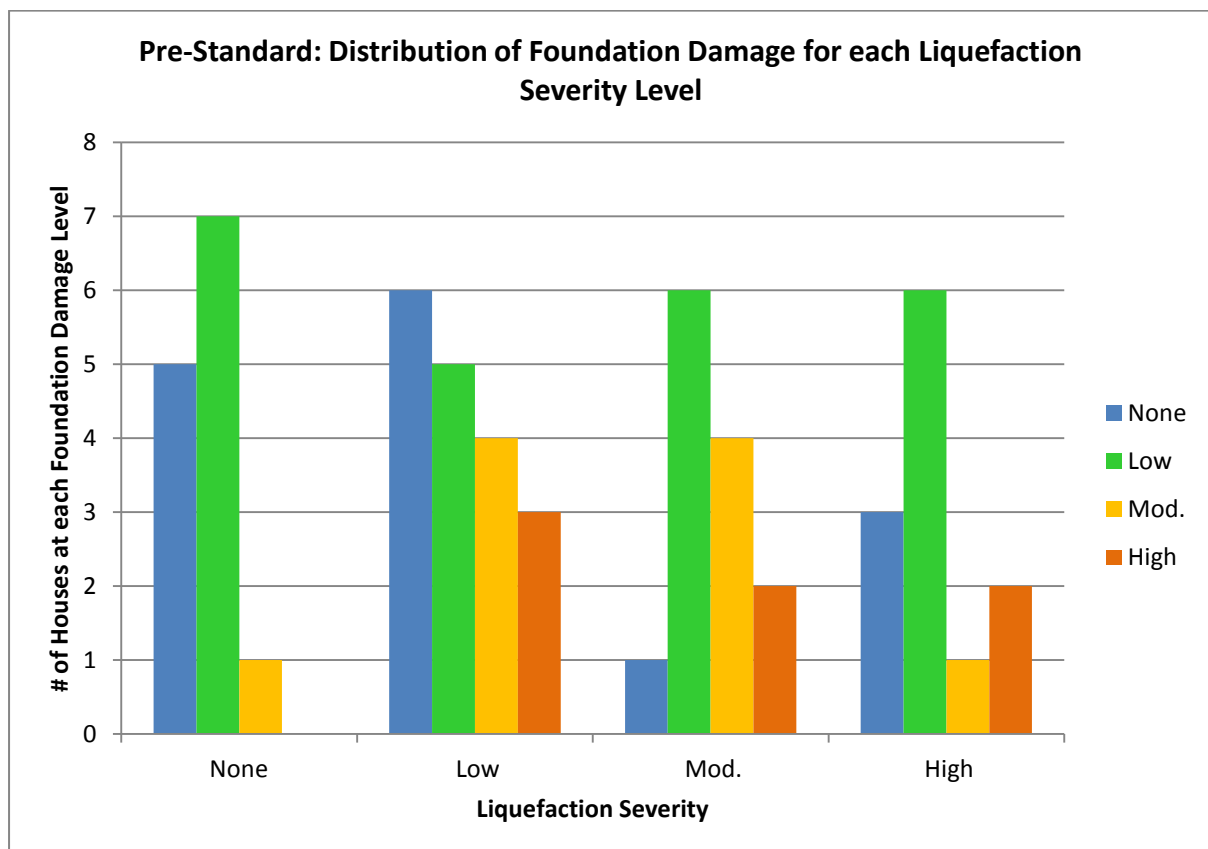


Figure A. 16. Distribution of foundation damage level for differing levels of liquefaction severity for all pre-standard houses.

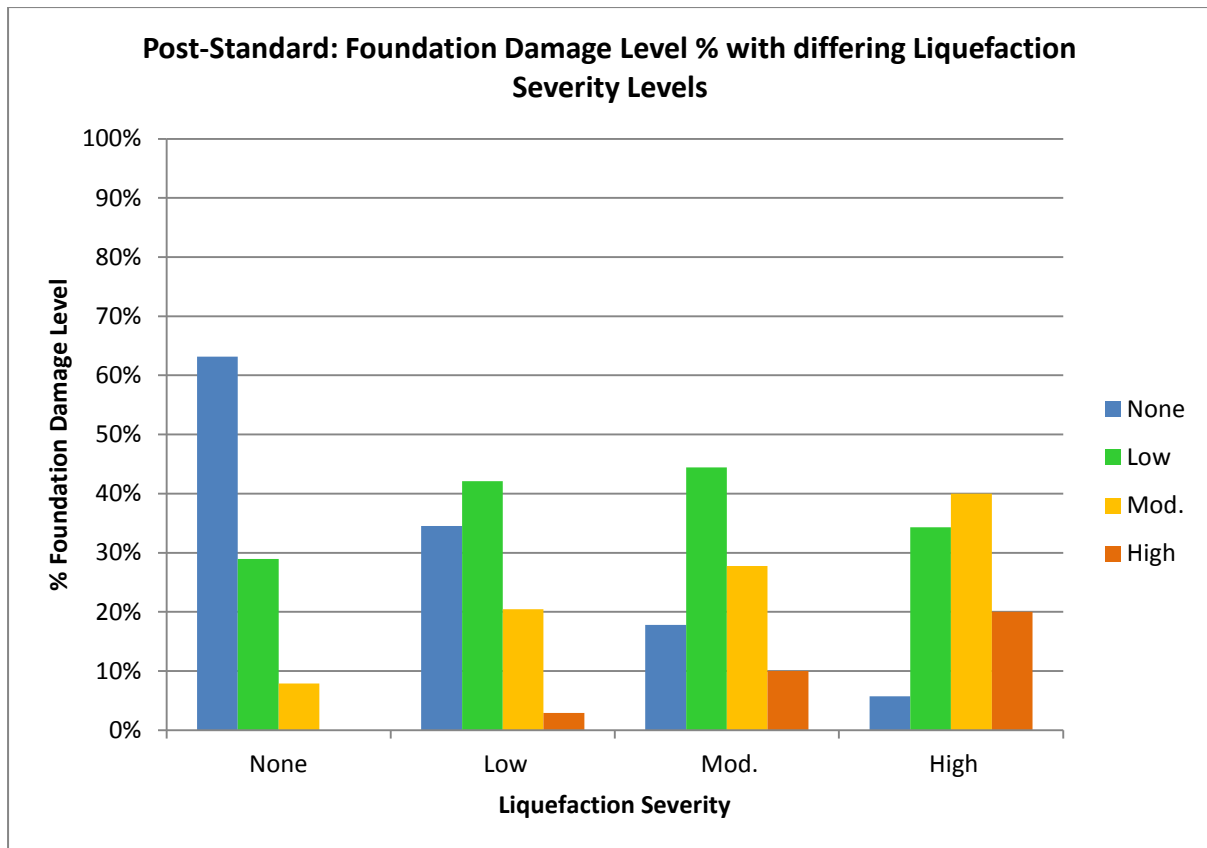


Figure A. 17. Distribution of foundation damage level in percentage for differing levels of liquefaction severity for all post-standard houses.

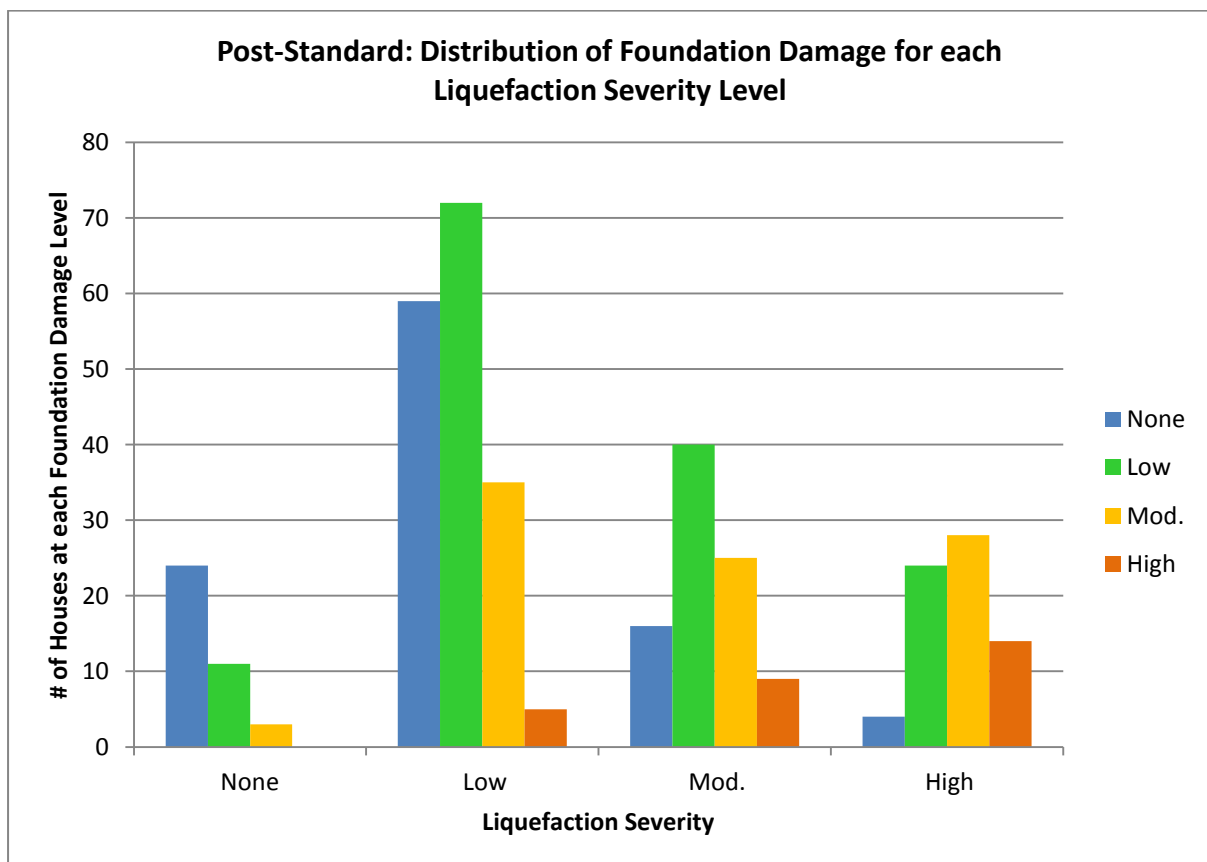


Figure A. 18. Distribution of foundation damage level for differing levels of liquefaction severity for all post-standard houses.

E.1.3: By Building Weight

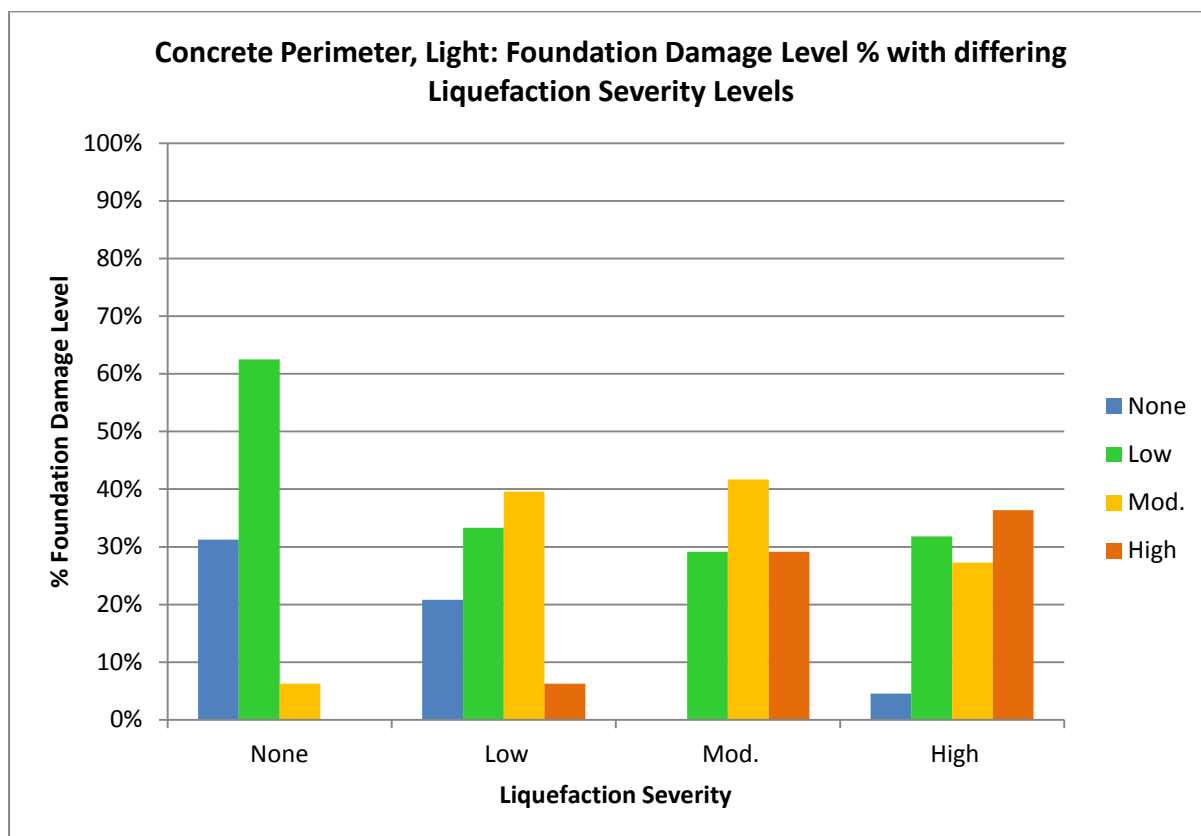


Figure A. 19. Distribution of foundation damage level in percentage for differing levels of liquefaction severity for light houses on concrete perimeter foundations.

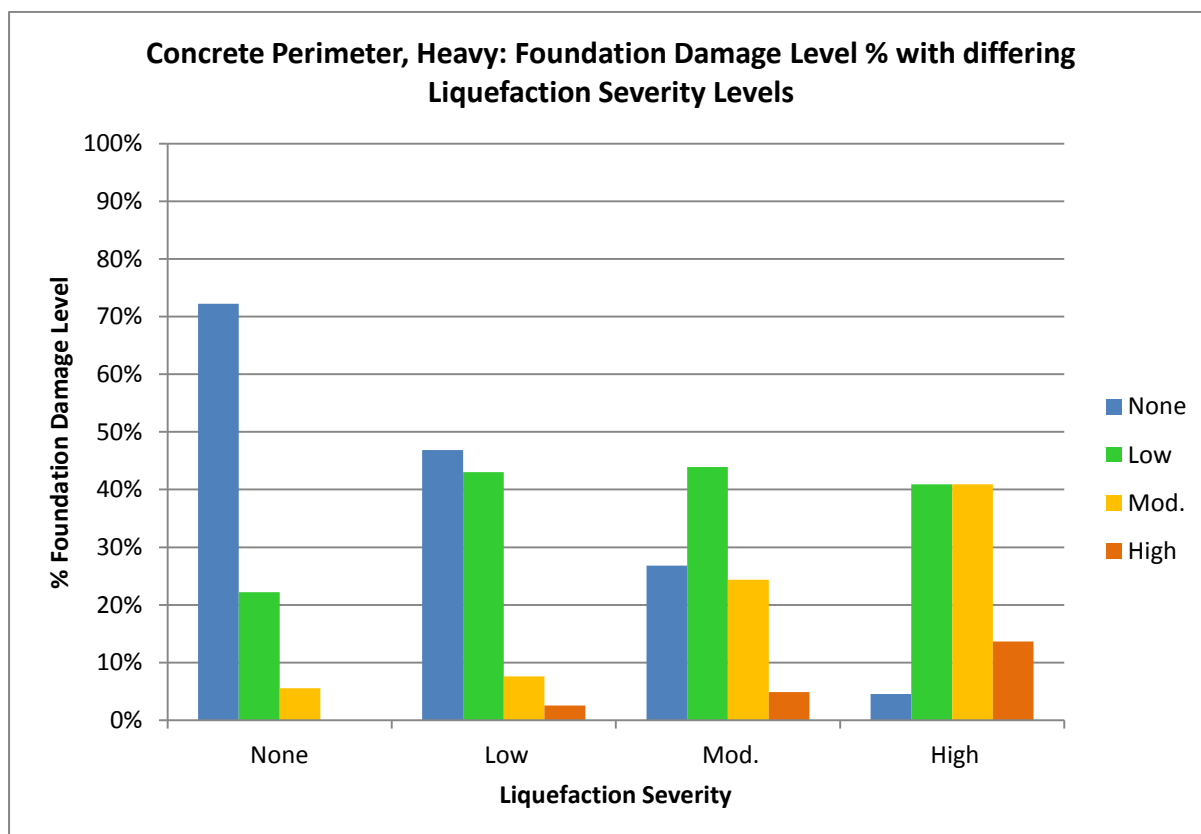


Figure A. 20. Distribution of foundation damage level in percentage for differing levels of liquefaction severity for heavy houses on concrete perimeter foundations.

E.2: LiDAR Ground Settlement vs. Foundation Damage

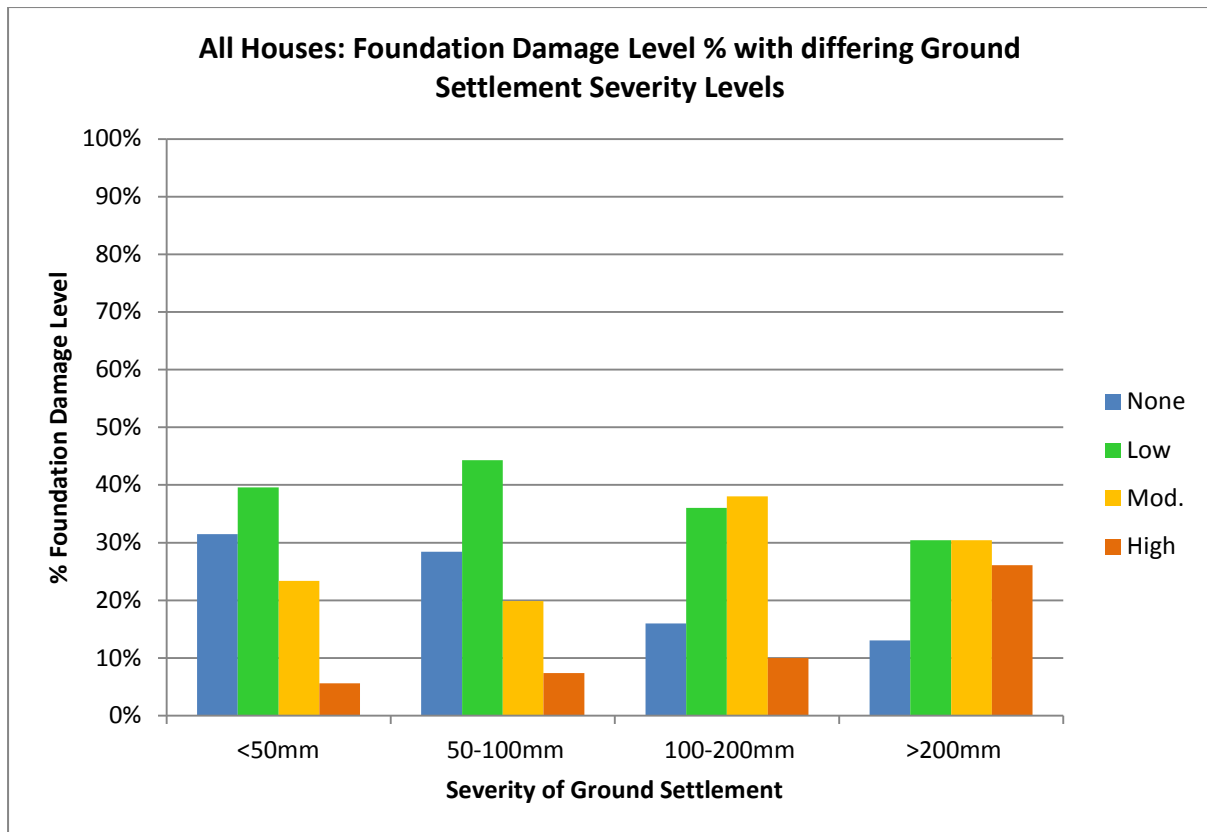


Figure A. 21. Distribution of foundation damage level in percentage for differing degrees of ground settlement for all houses.

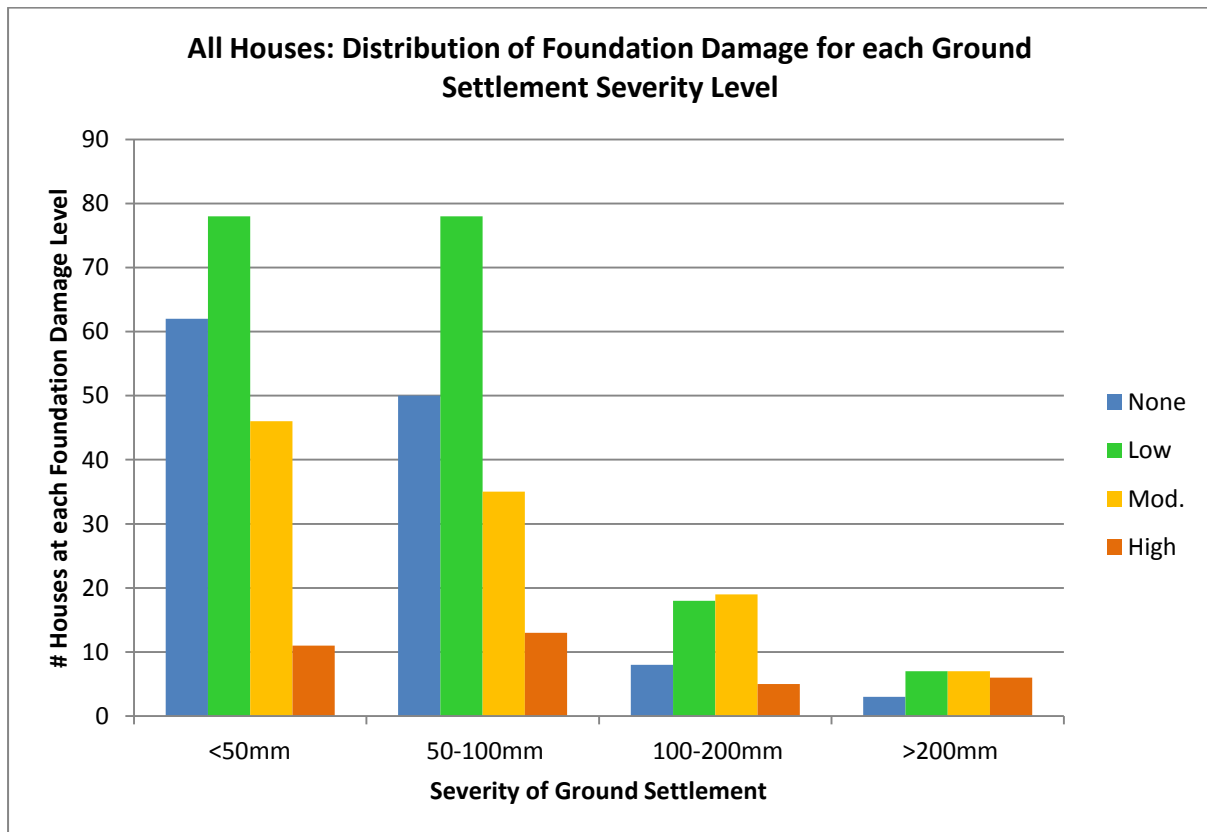


Figure A. 22. Distribution of foundation damage level for differing degrees of ground settlement for all houses.

E.2.1: By Foundation Type

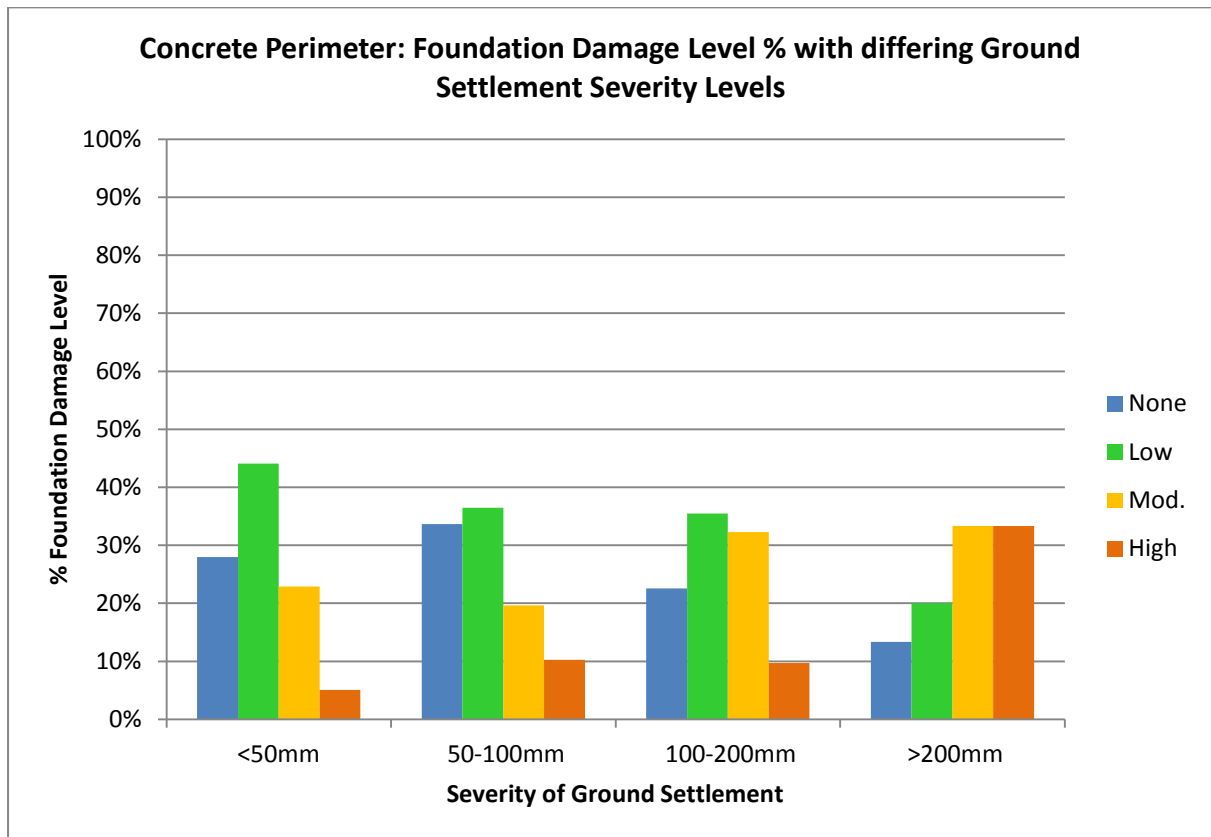


Figure A. 23. Distribution of foundation damage level in percentage for differing degrees of ground settlement for concrete perimeter foundations.

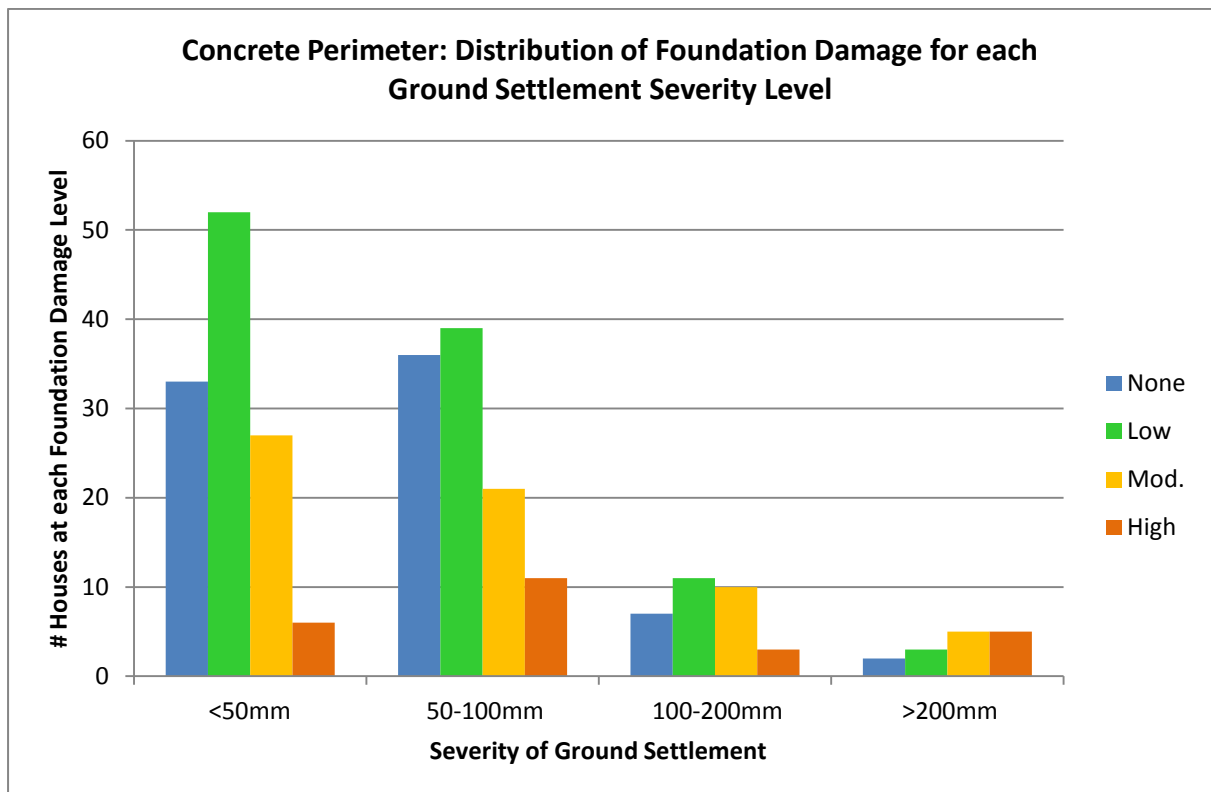


Figure A. 24. Distribution of foundation damage level for differing degrees of ground settlement for concrete perimeter foundations.

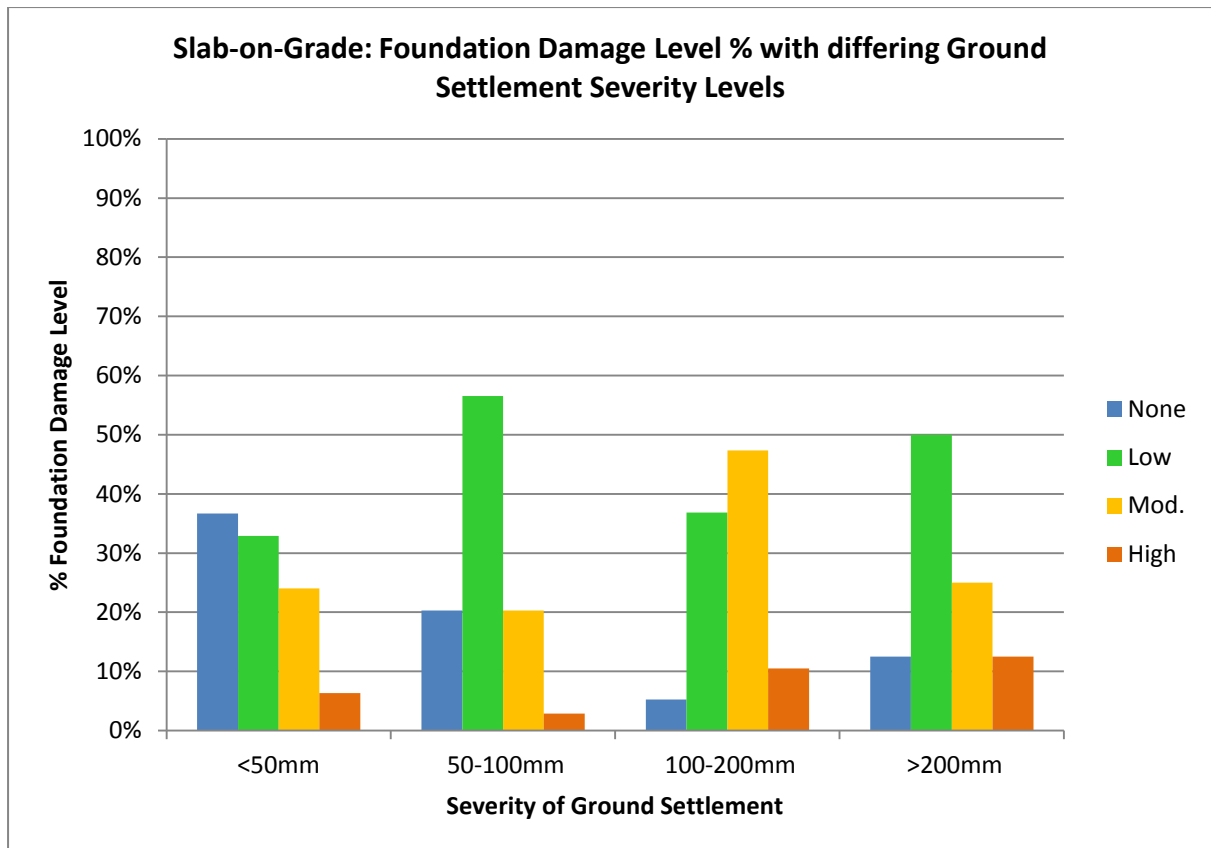


Figure A. 25. Distribution of foundation damage level in percentage for differing degrees of ground settlement for slab-on-grade foundations.

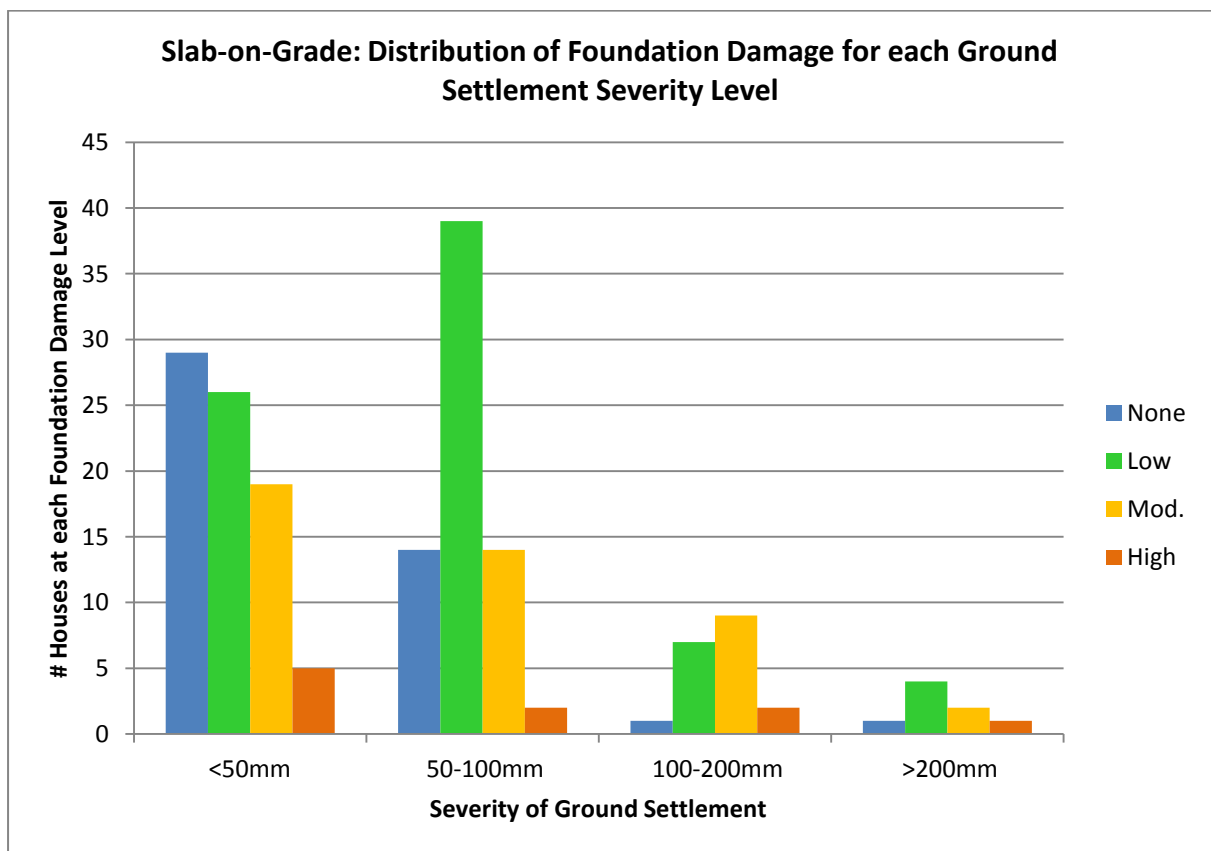


Figure A. 26. Distribution of foundation damage level for differing degrees of ground settlement for slab-on-grade foundations.

E.2.2: By Age

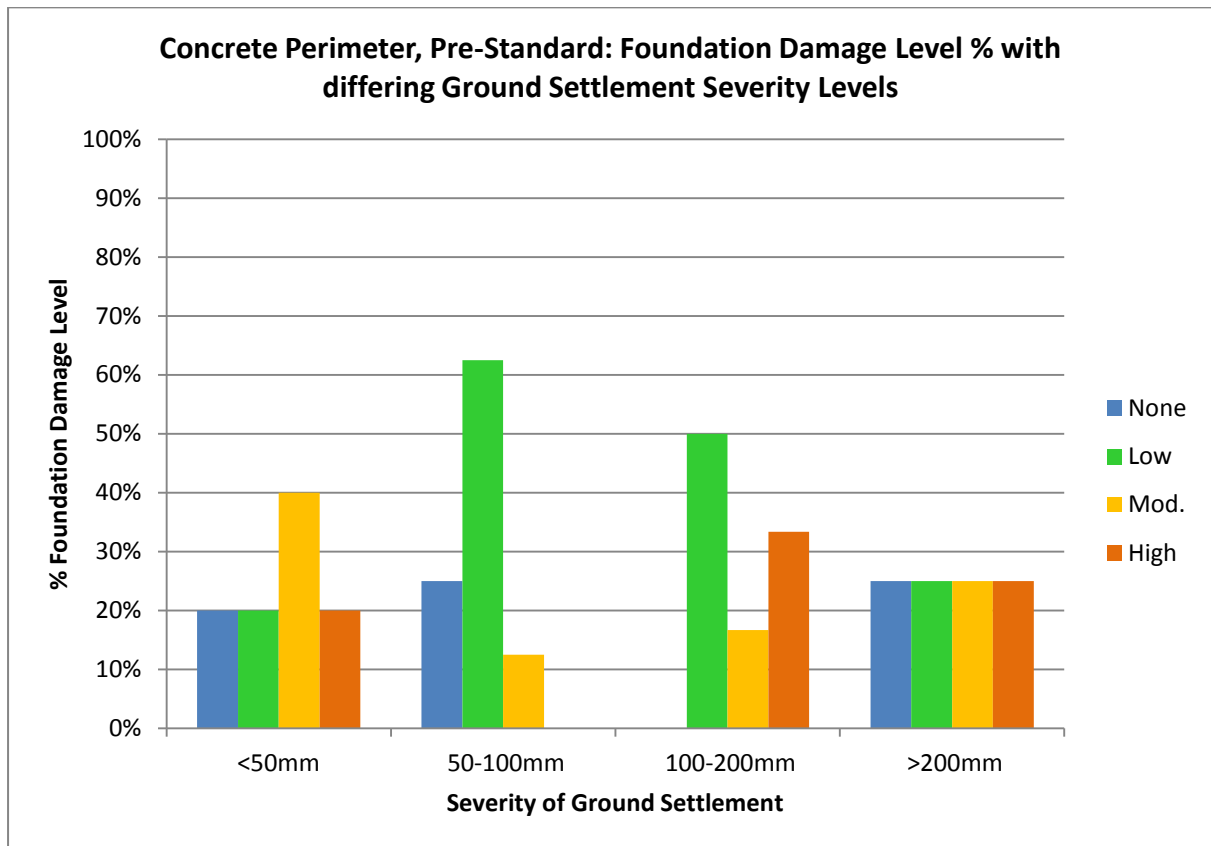


Figure A. 27. Distribution of foundation damage level in percentage for differing degrees of ground settlement for pre-standard, concrete perimeter foundations.

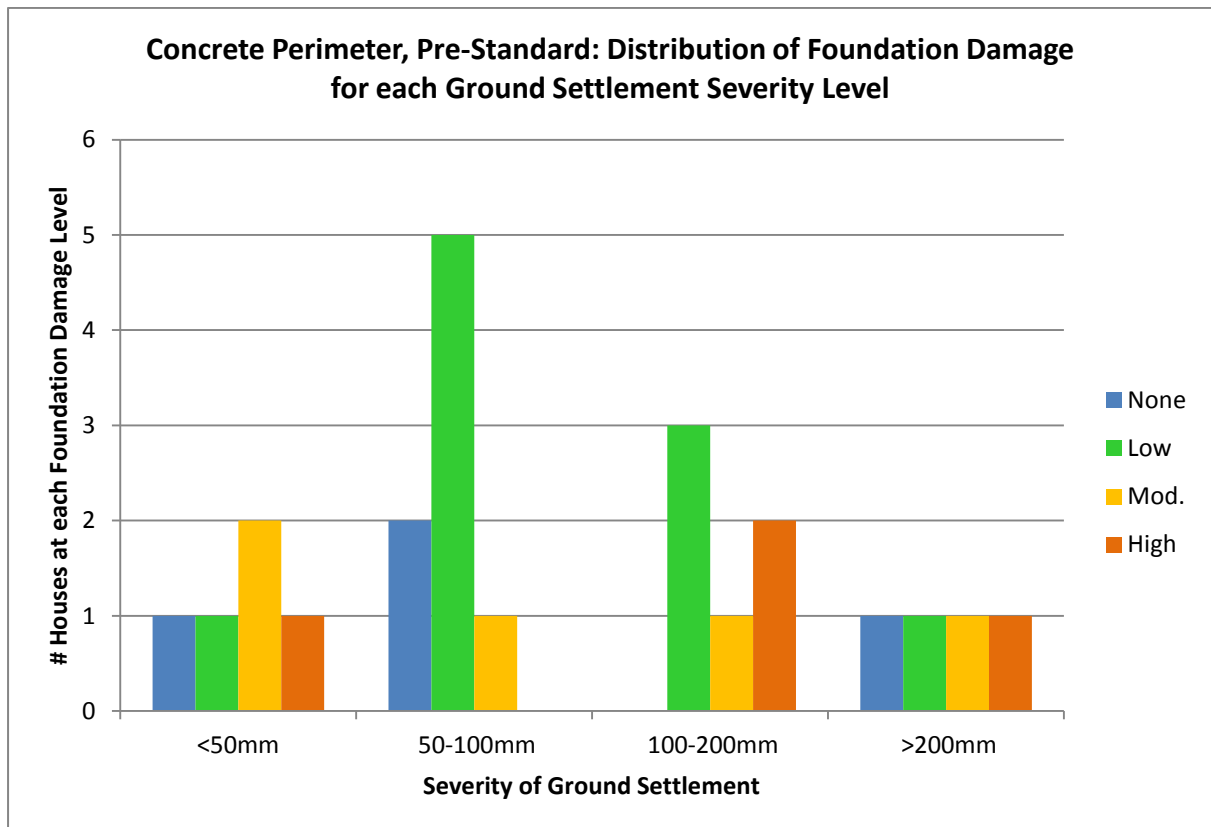


Figure A. 28. Distribution of foundation damage level for differing degrees of ground settlement for pre-standard, concrete perimeter foundations.

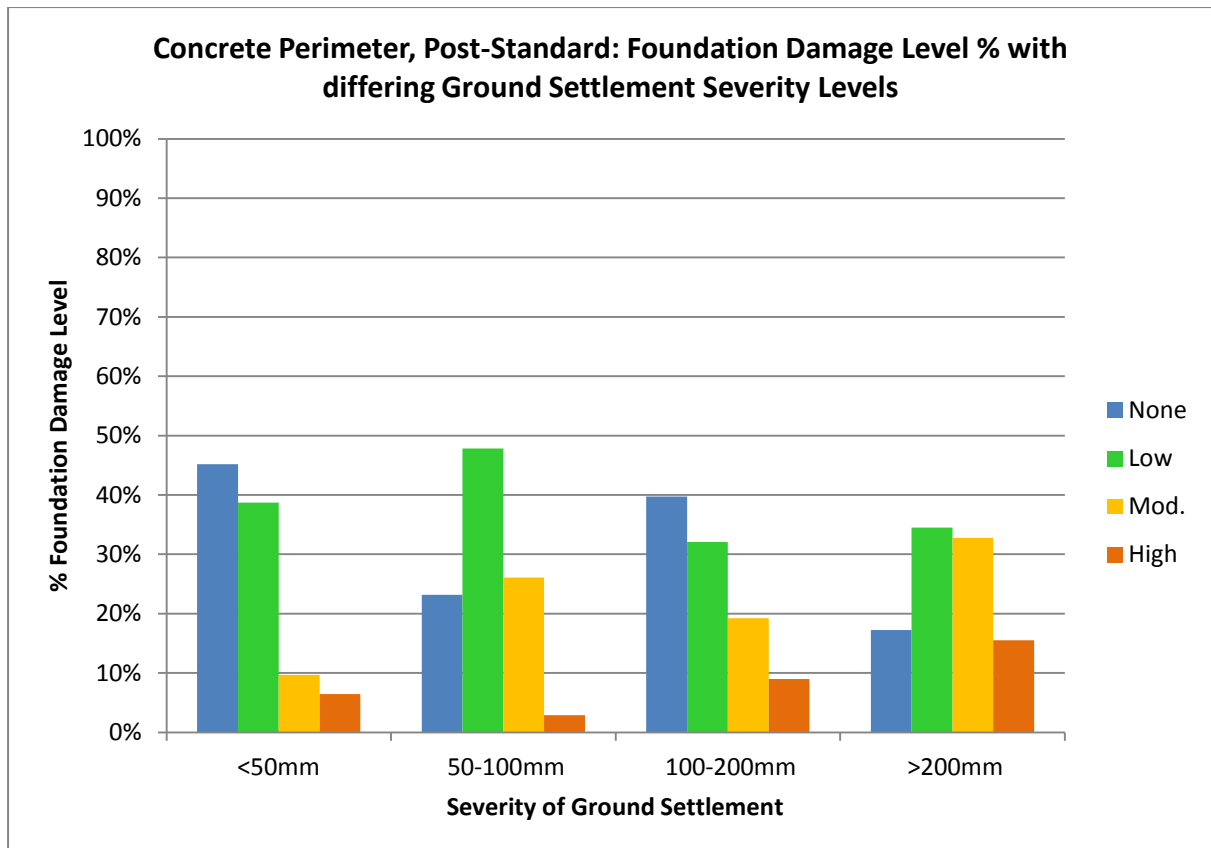


Figure A. 29. Distribution of foundation damage level in percentage for differing degrees of ground settlement for post-standard, concrete perimeter foundations.

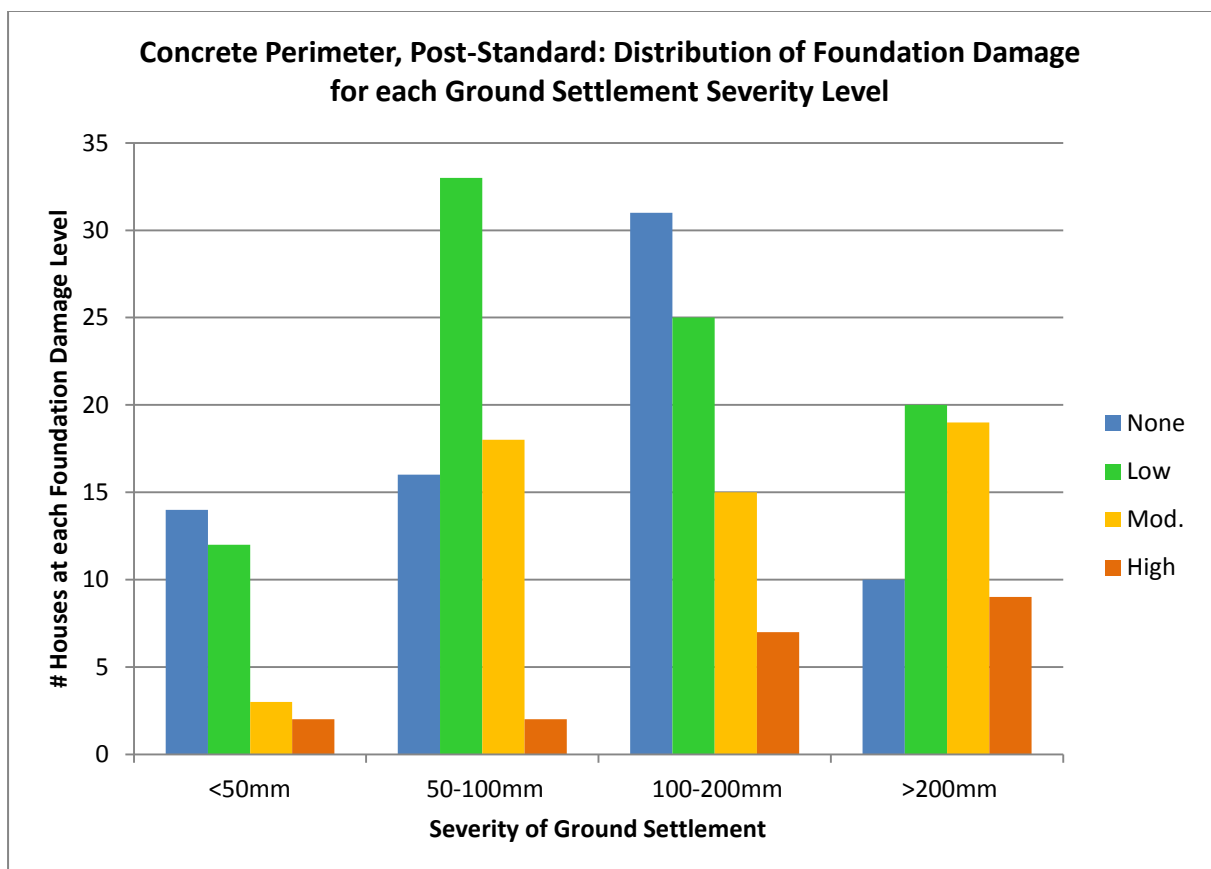


Figure A. 30. Distribution of foundation damage level for differing degrees of ground settlement for post-standard, concrete perimeter foundations.

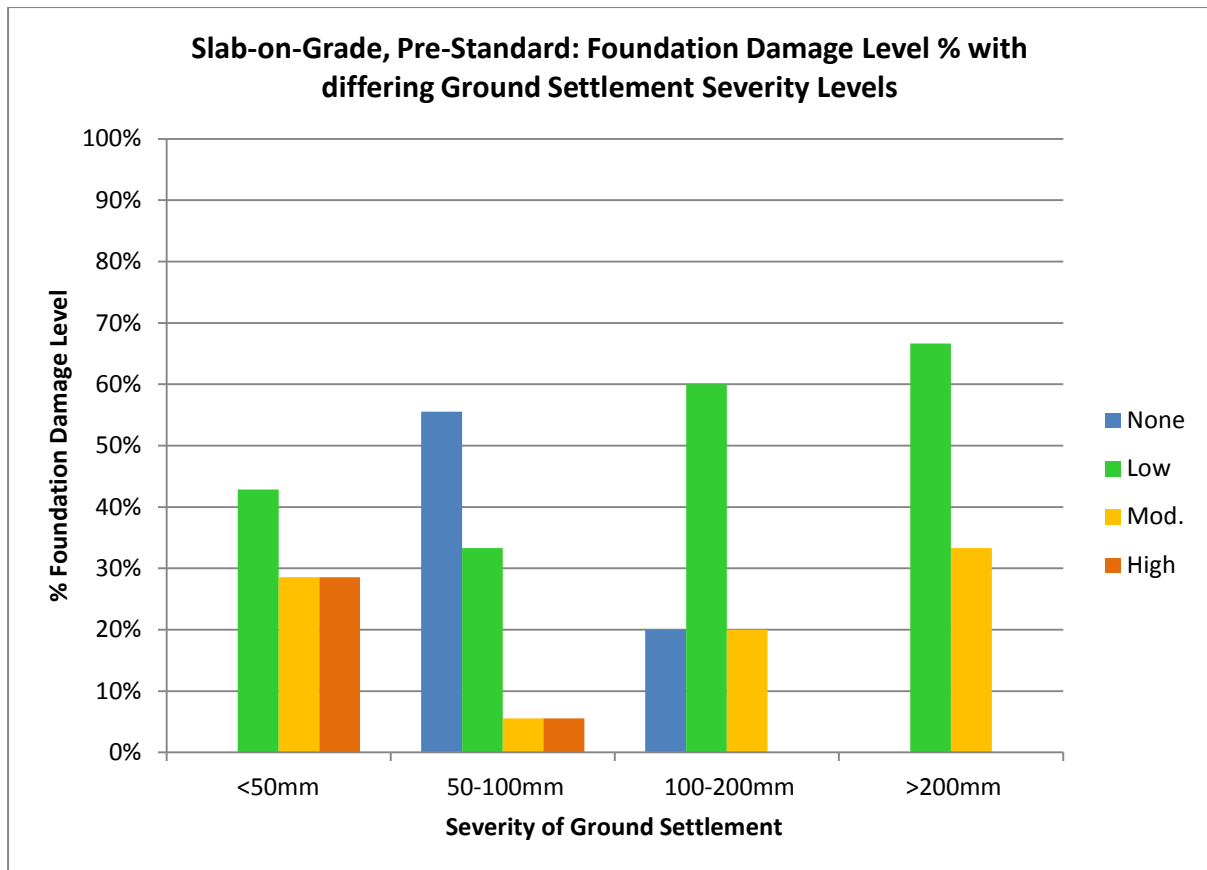


Figure A. 31. Distribution of foundation damage level in percentage for differing degrees of ground settlement for pre-standard, slab-on-grade foundations.

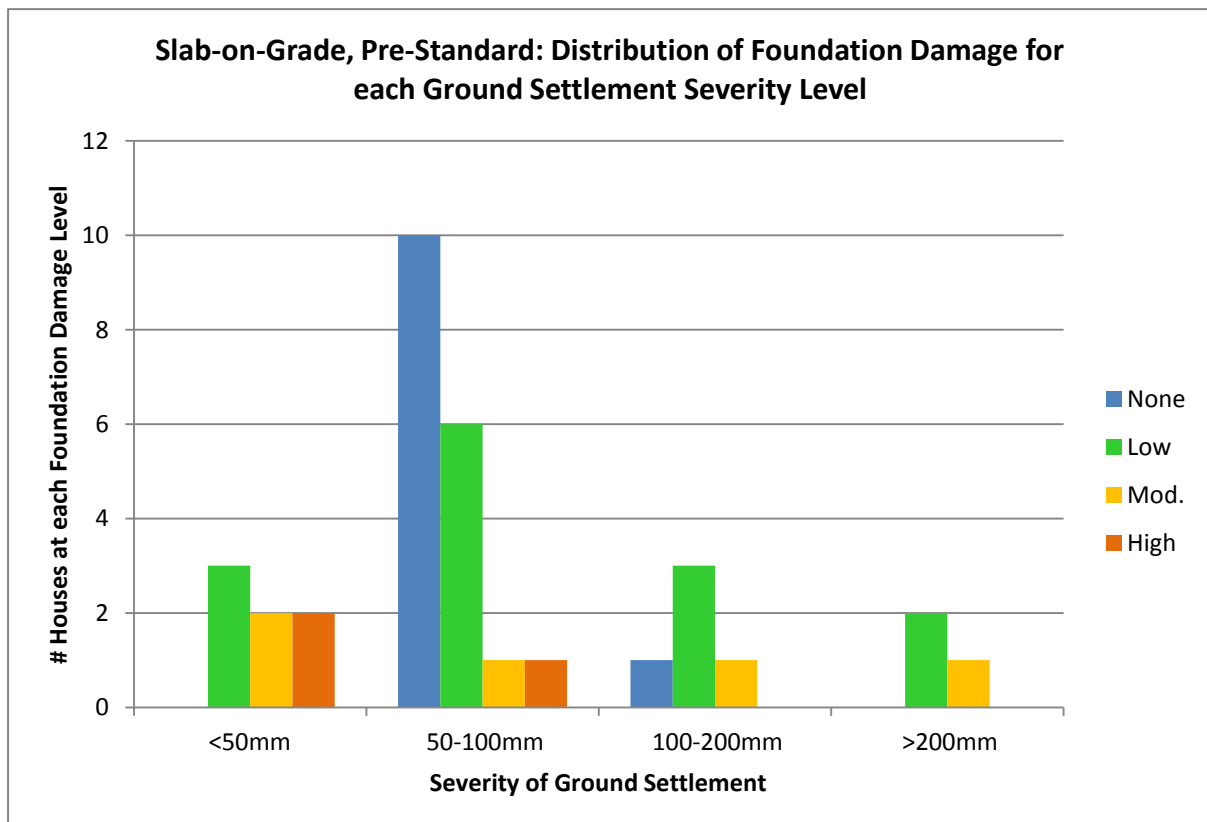


Figure A. 32. Distribution of foundation damage level for differing degrees of ground settlement for pre-standard, slab-on-grade foundations.

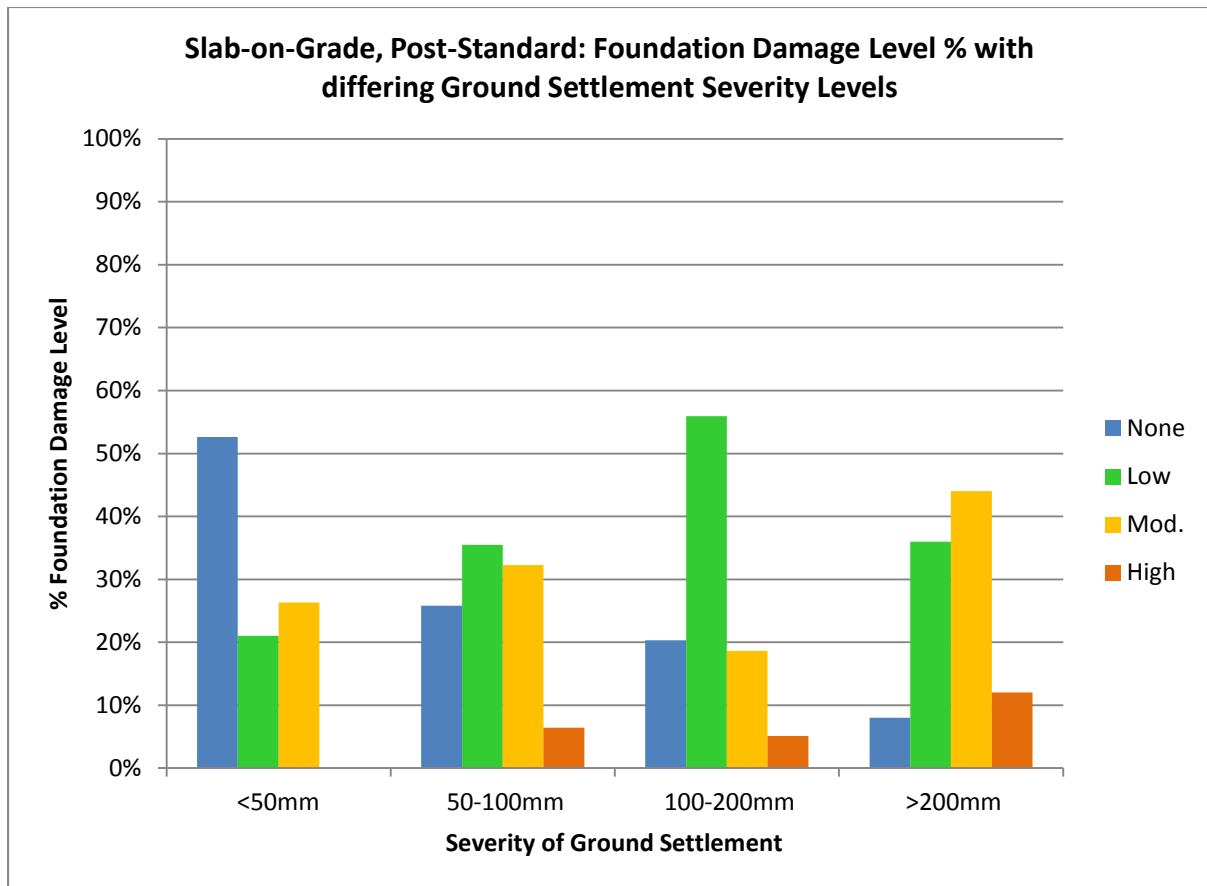


Figure A. 33. Distribution of foundation damage level in percentage for differing degrees of ground settlement for post-standard, slab-on-grade foundations.

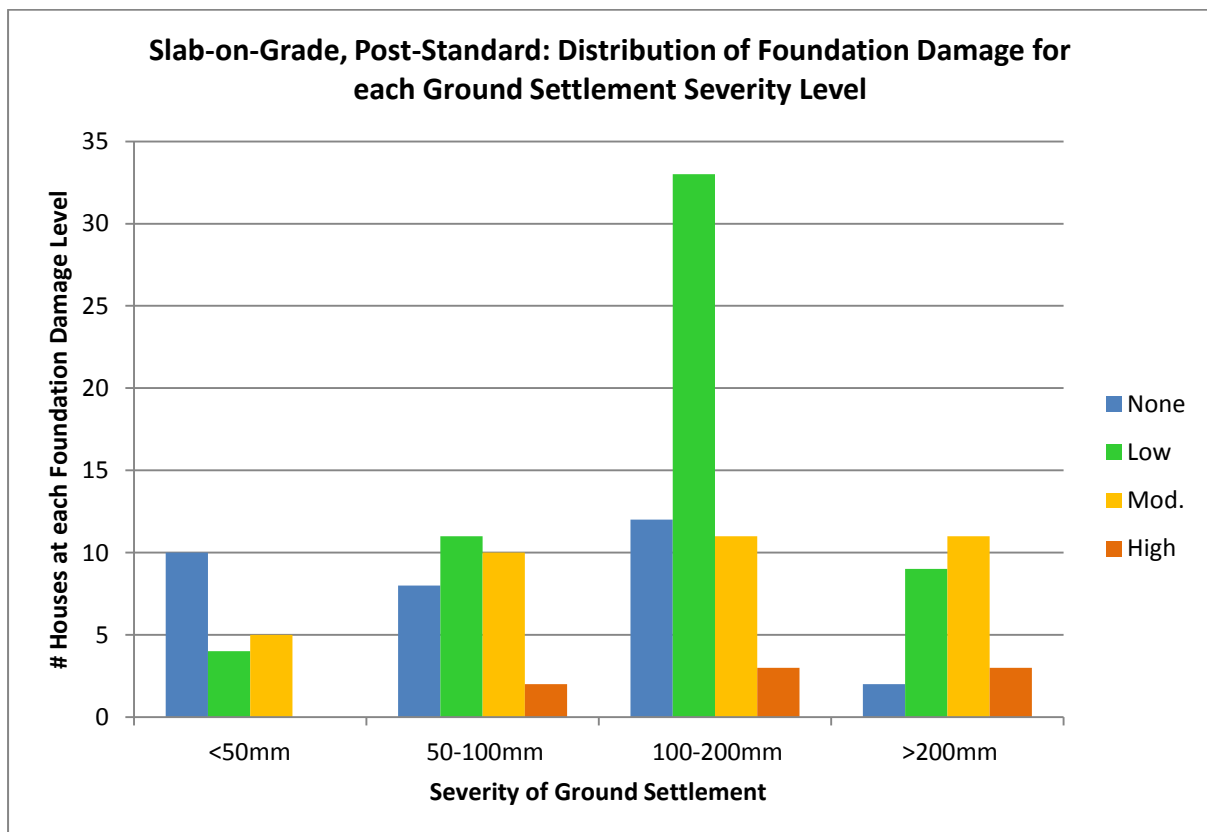


Figure A. 34. Distribution of foundation damage level for differing degrees of ground settlement for post-standard, slab-on-grade foundations.

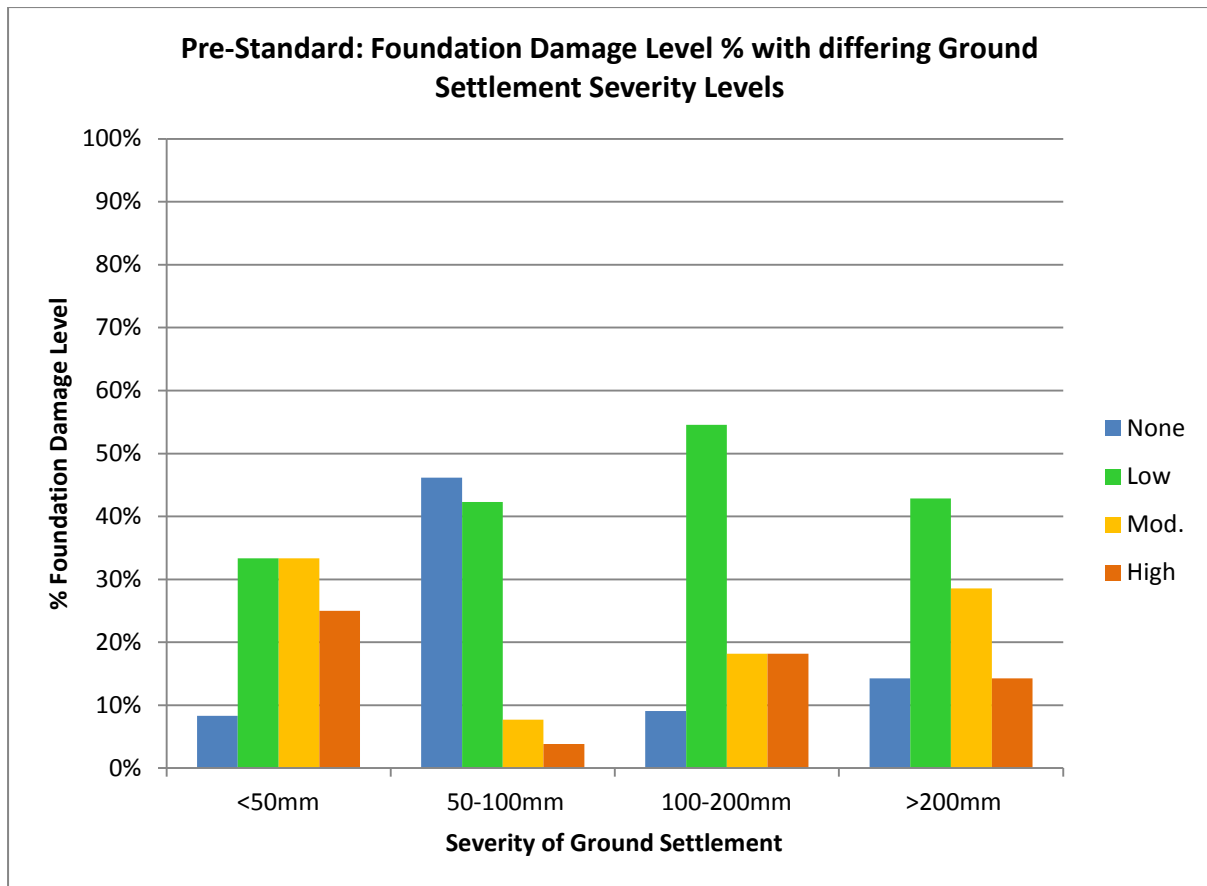


Figure A. 35. Distribution of foundation damage level in percentage for differing degrees of ground settlement for all pre-standard foundations.

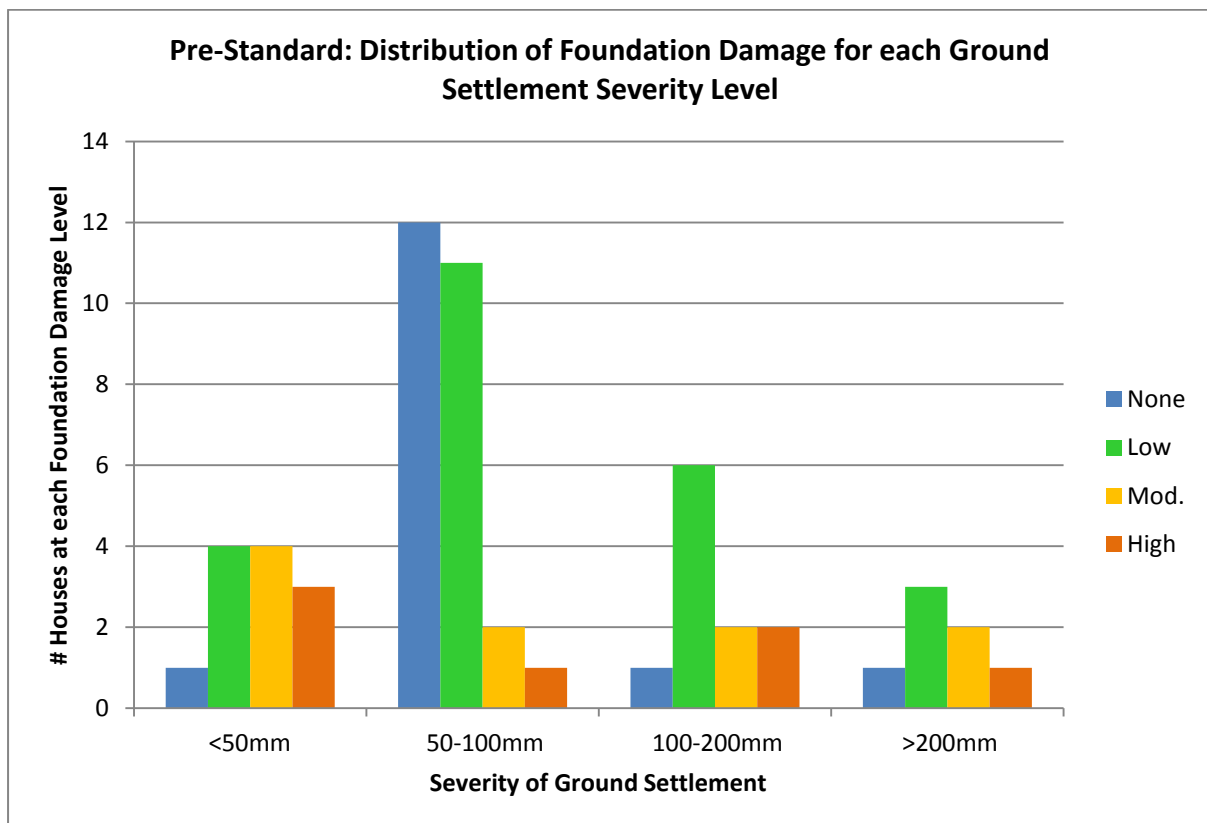


Figure A. 36. Distribution of foundation damage level for differing degrees of ground settlement for all pre-standard foundations.

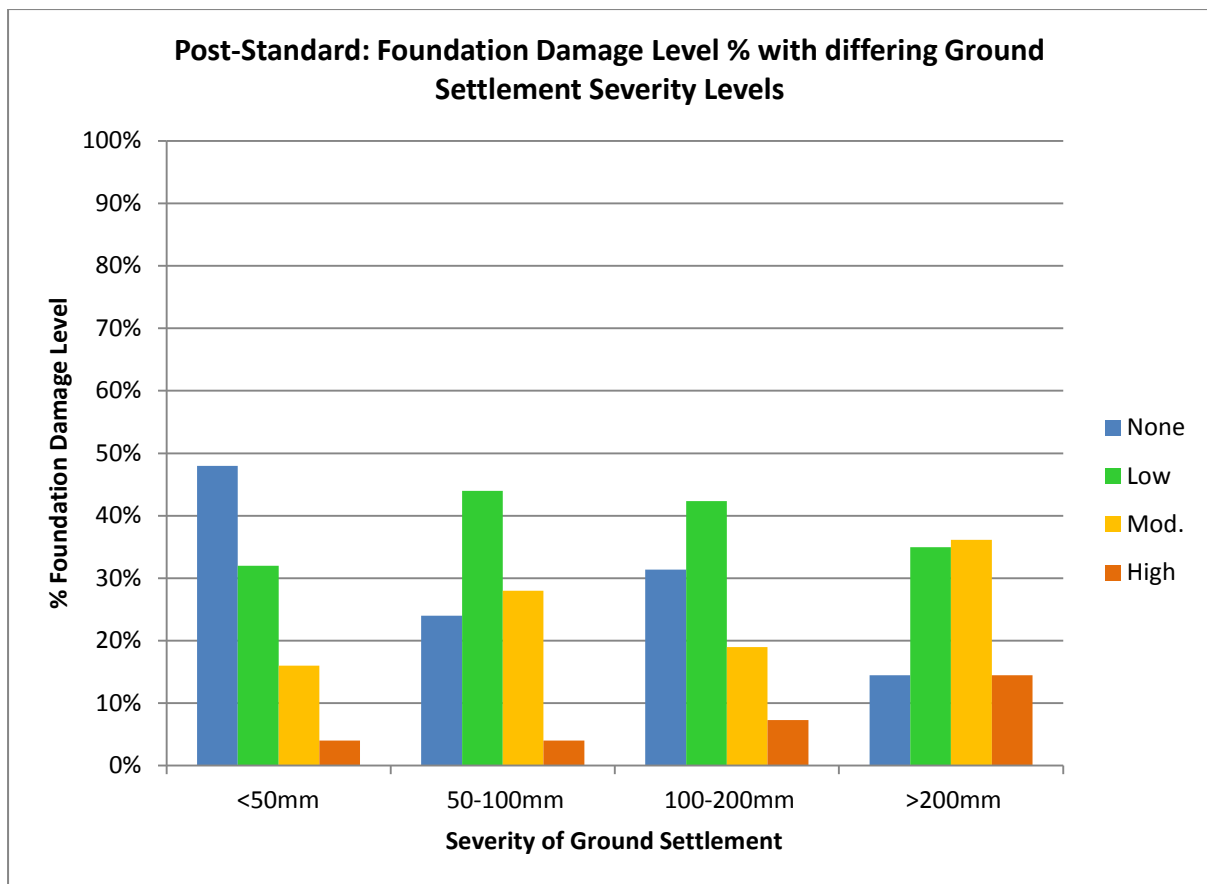


Figure A. 37. Distribution of foundation damage level in percentage for differing degrees of ground settlement for all post-standard foundations.

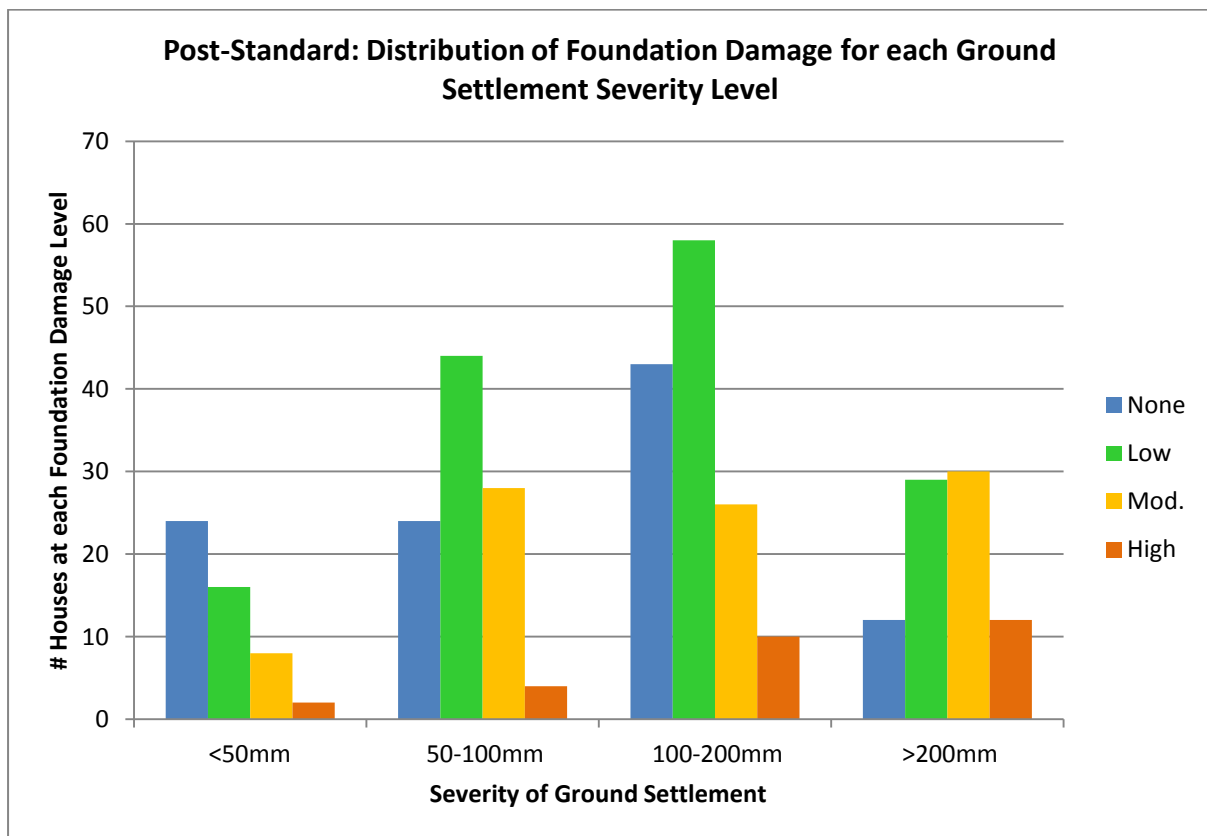


Figure A. 38. Distribution of foundation damage level for differing degrees of ground settlement for all post-standard foundations.

E.2.3: By Building Weight

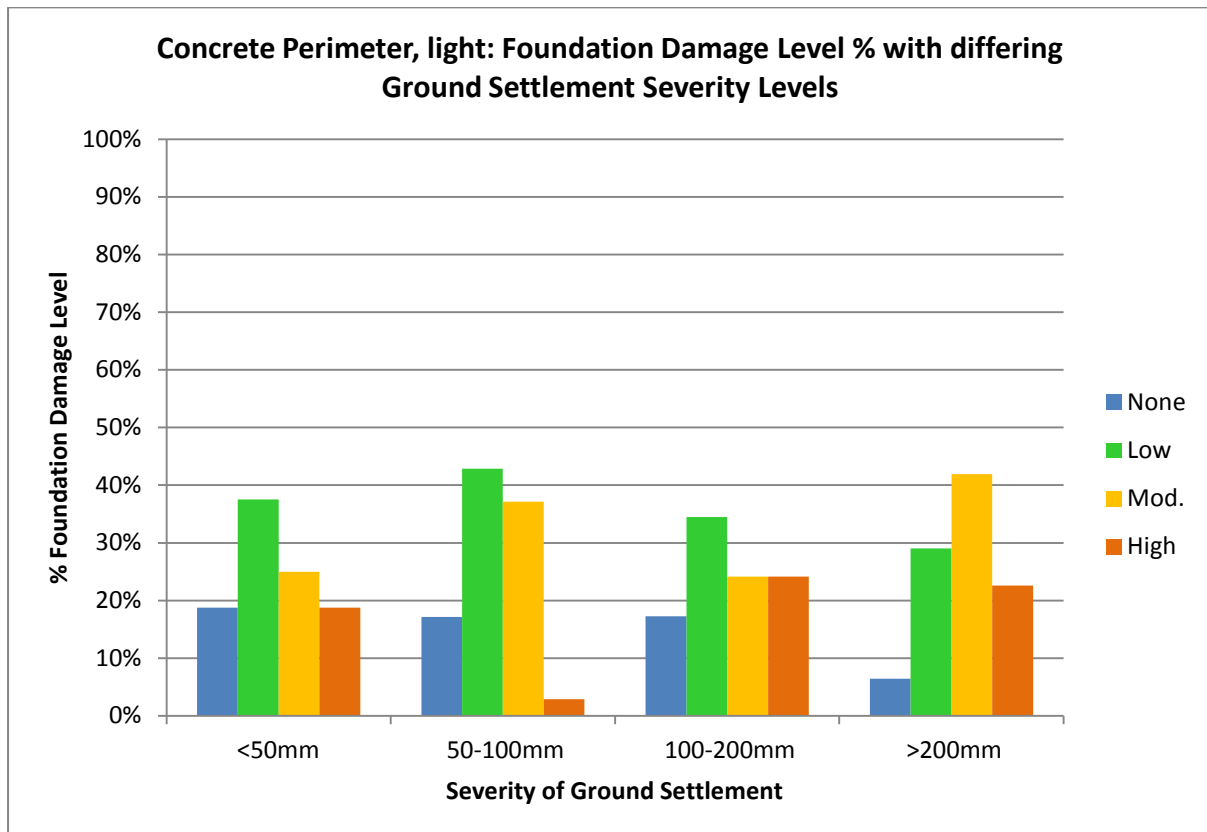


Figure A. 39. Distribution of foundation damage level in percentage for differing degrees of ground settlement for light buildings on concrete perimeter foundations.

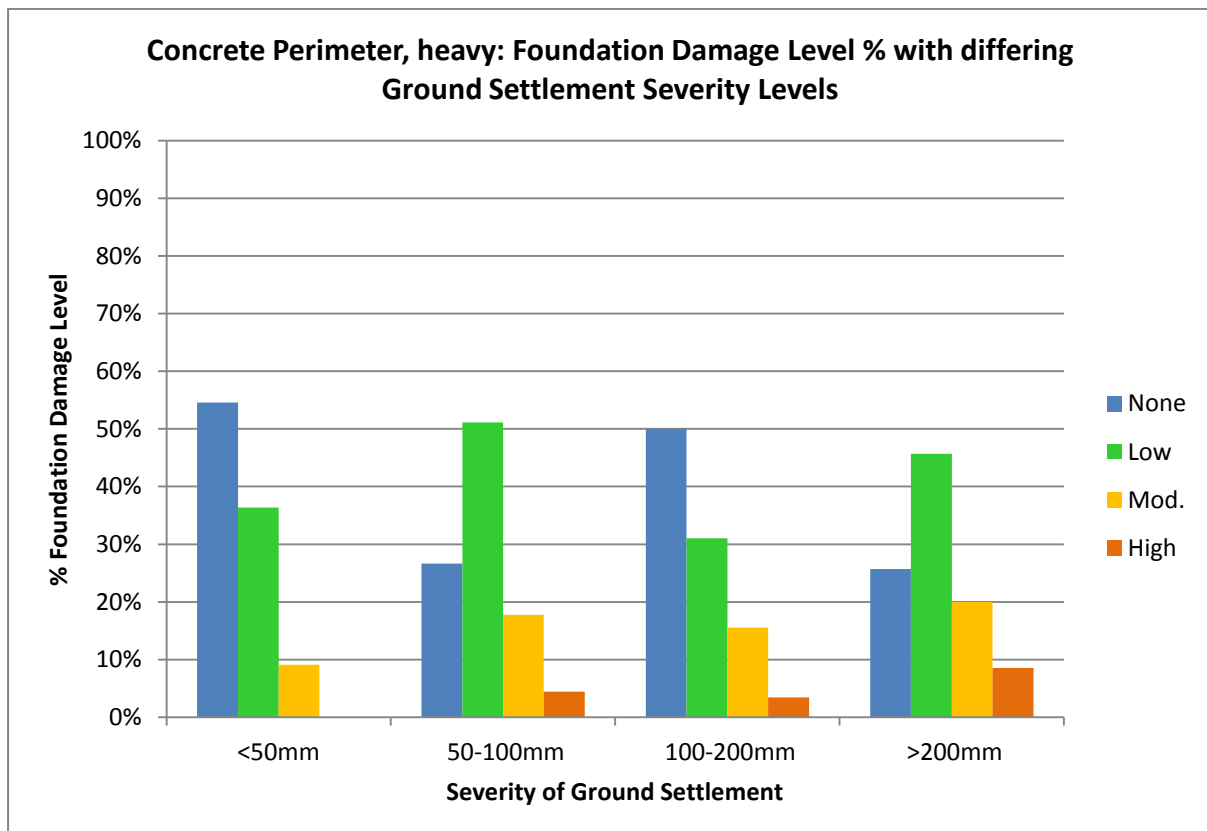


Figure A. 40. Distribution of foundation damage level in percentage for differing degrees of ground settlement for heavy buildings on concrete perimeter foundations.

E.3: LiDAR Lateral Spreading vs. Foundation Damage

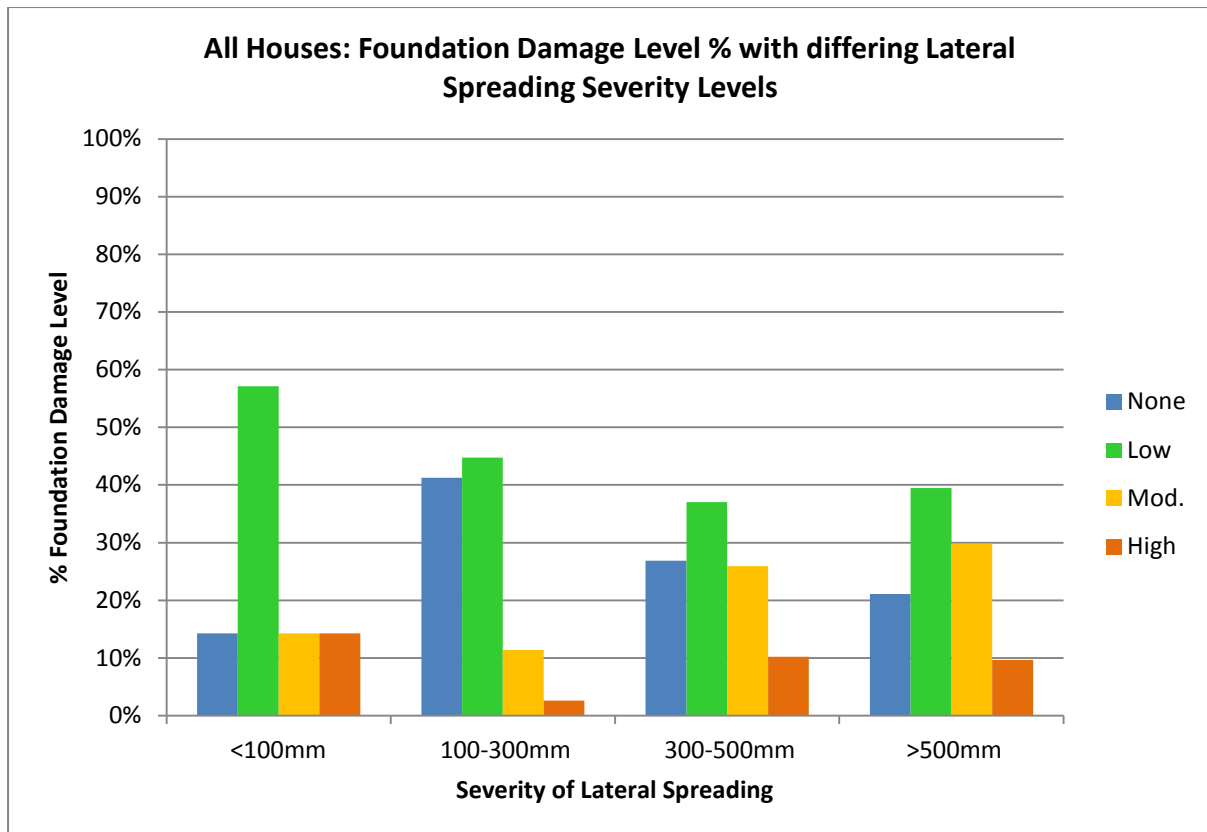


Figure A. 41. Distribution of foundation damage level in percentage for differing degrees of lateral ground movement for all inspected buildings.

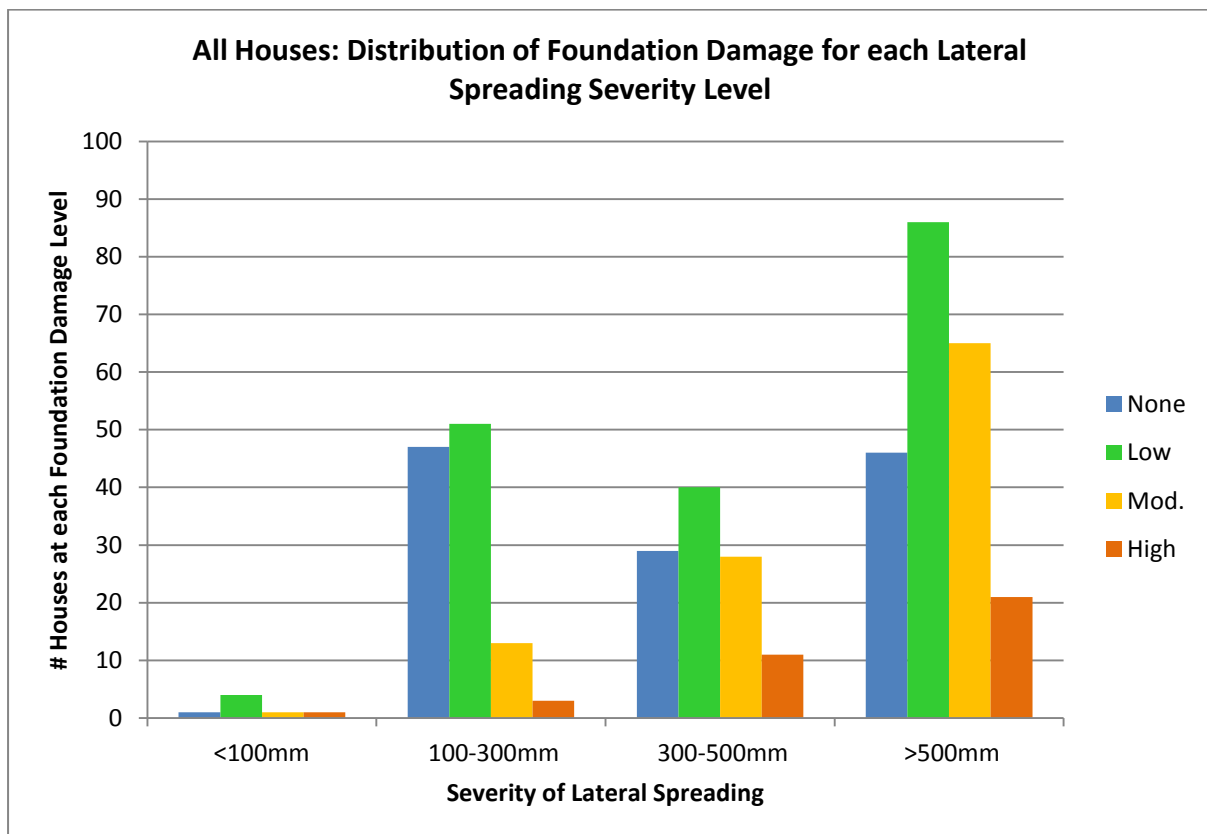


Figure A. 42. Distribution of foundation damage level for differing degrees of lateral ground movement for all inspected buildings.

E.3.1: By Foundation Type

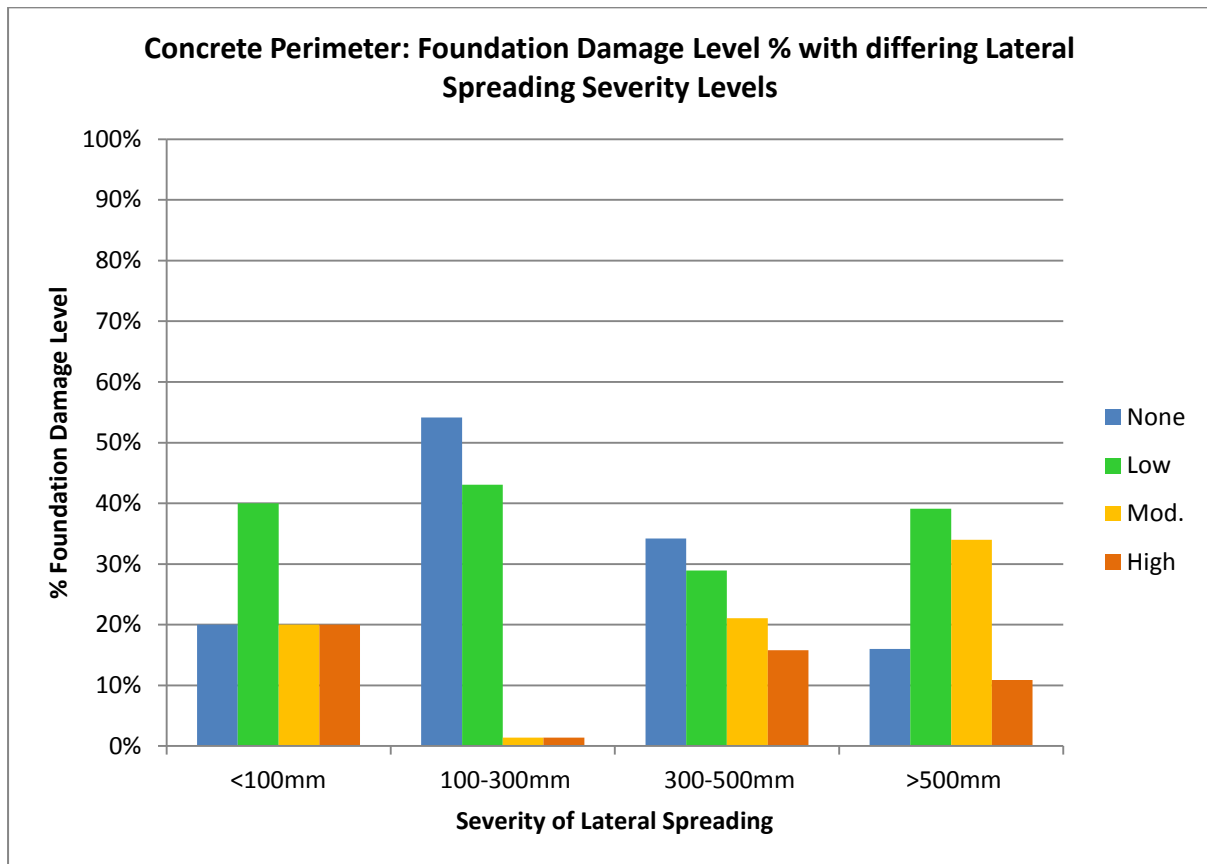


Figure A. 43. Distribution of foundation damage level in percentage for differing degrees of lateral ground movement for concrete perimeter foundations.

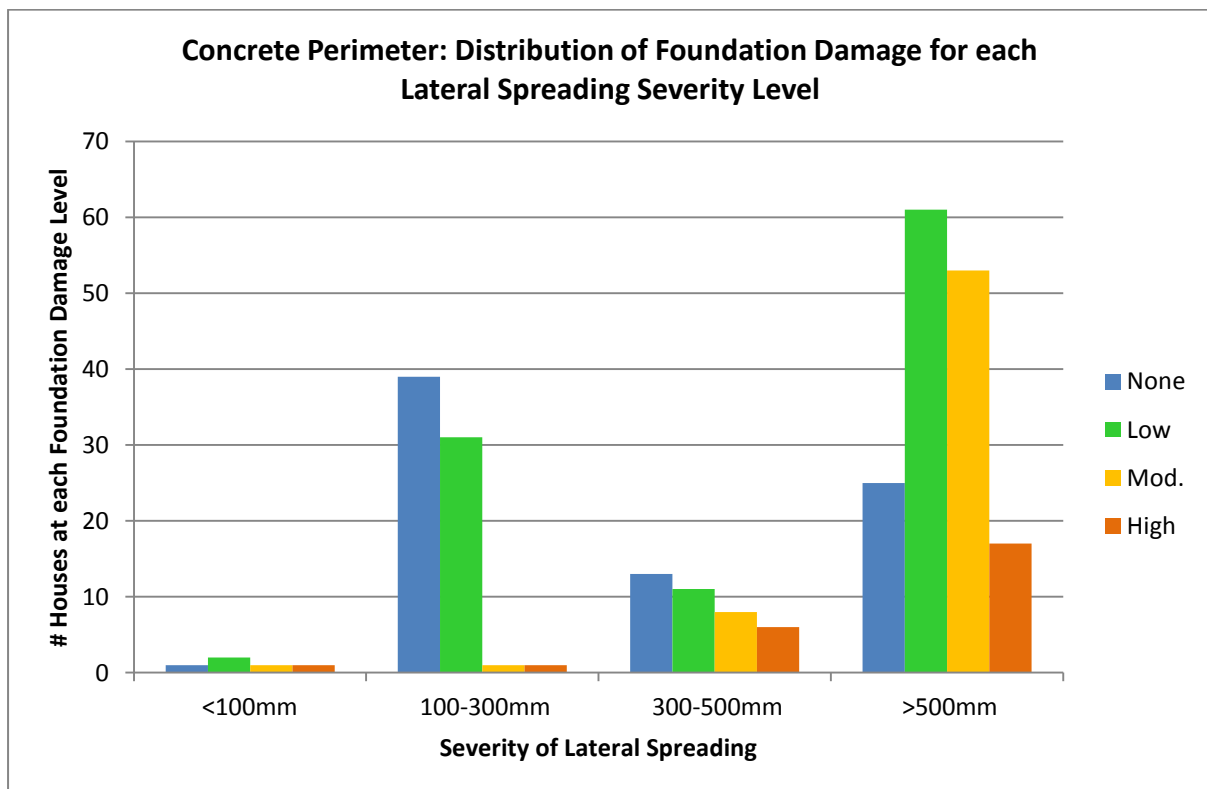


Figure A. 44. Distribution of foundation damage level for differing degrees of lateral ground movement for concrete perimeter foundations.

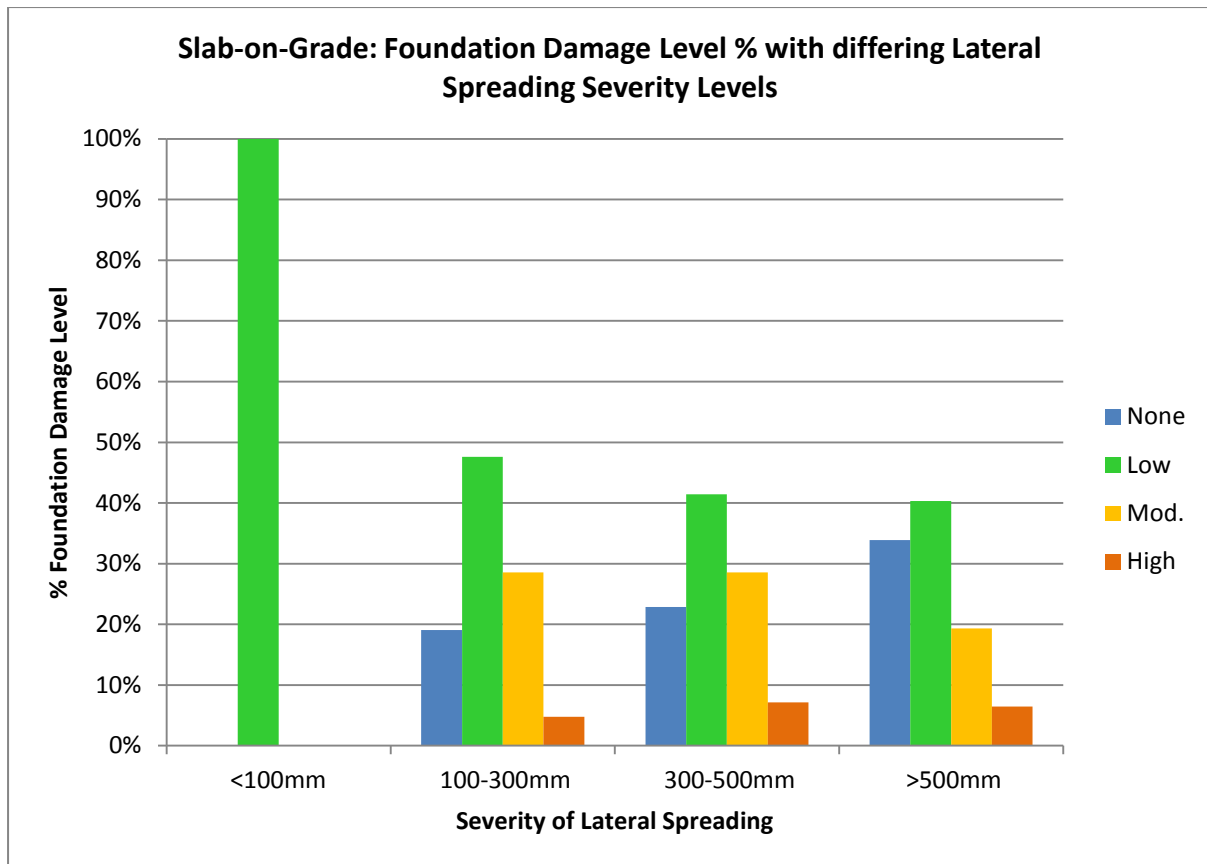


Figure A. 45. Distribution of foundation damage level in percentage for differing degrees of lateral ground movement for slab-on-grade foundations.

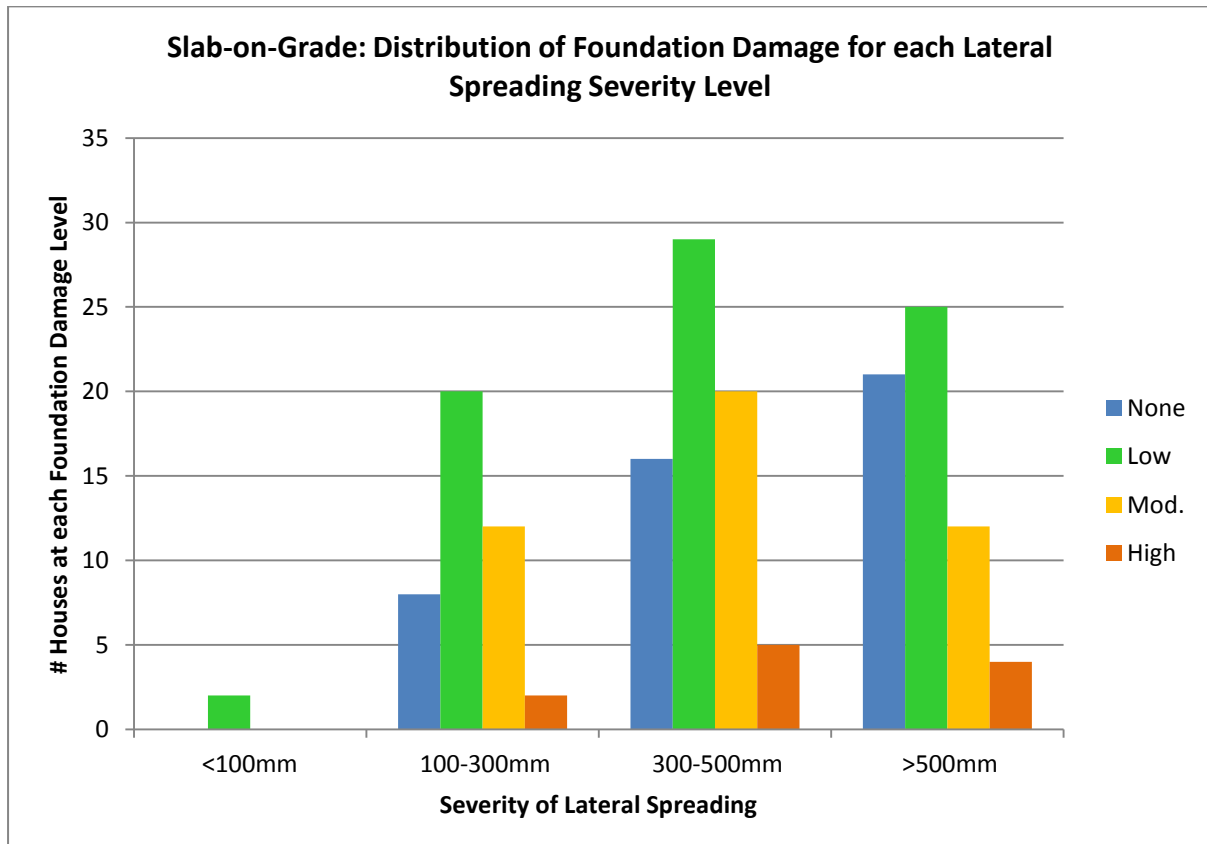


Figure A. 46. Distribution of foundation damage level for differing degrees of lateral ground movement for slab-on-grade foundations.

E.3.2: By Age

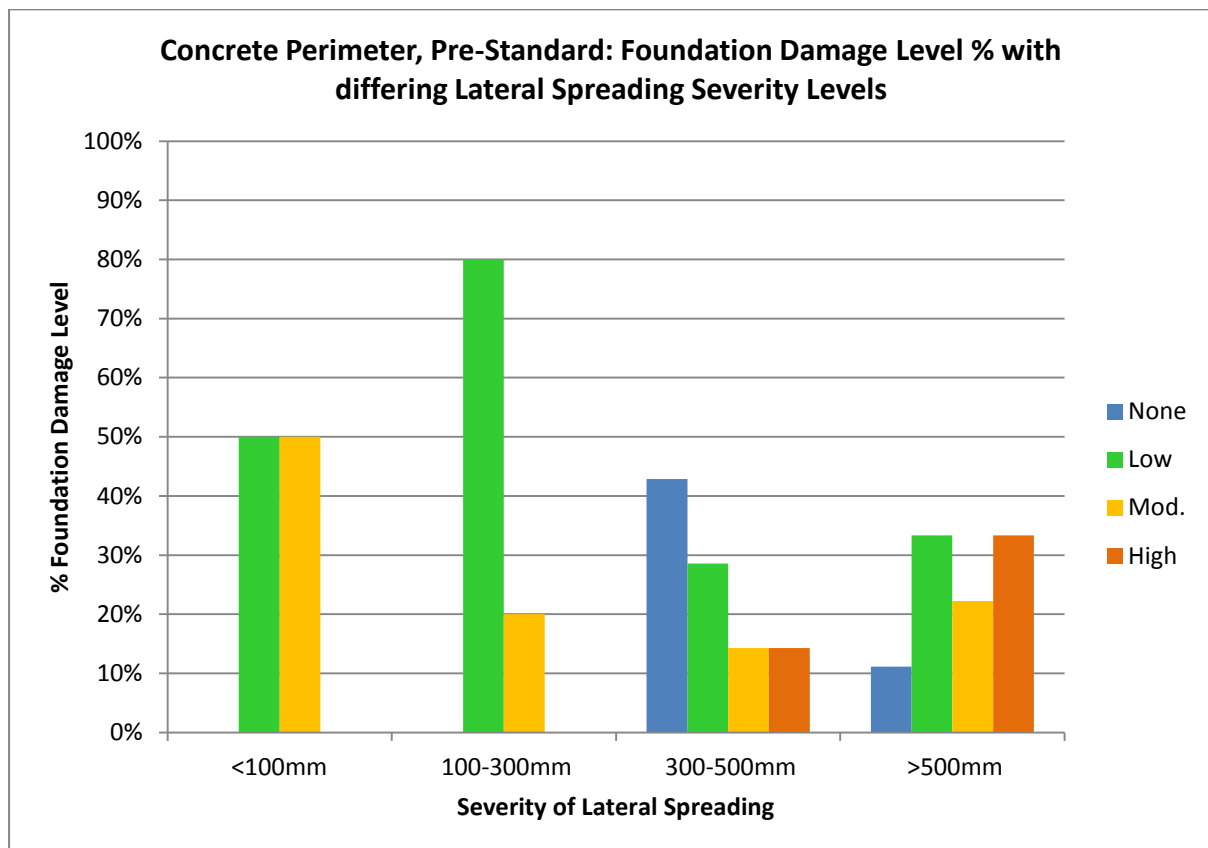


Figure A. 47. Distribution of foundation damage level in percentage for differing degrees of lateral ground movement for pre-standard, concrete perimeter foundations.

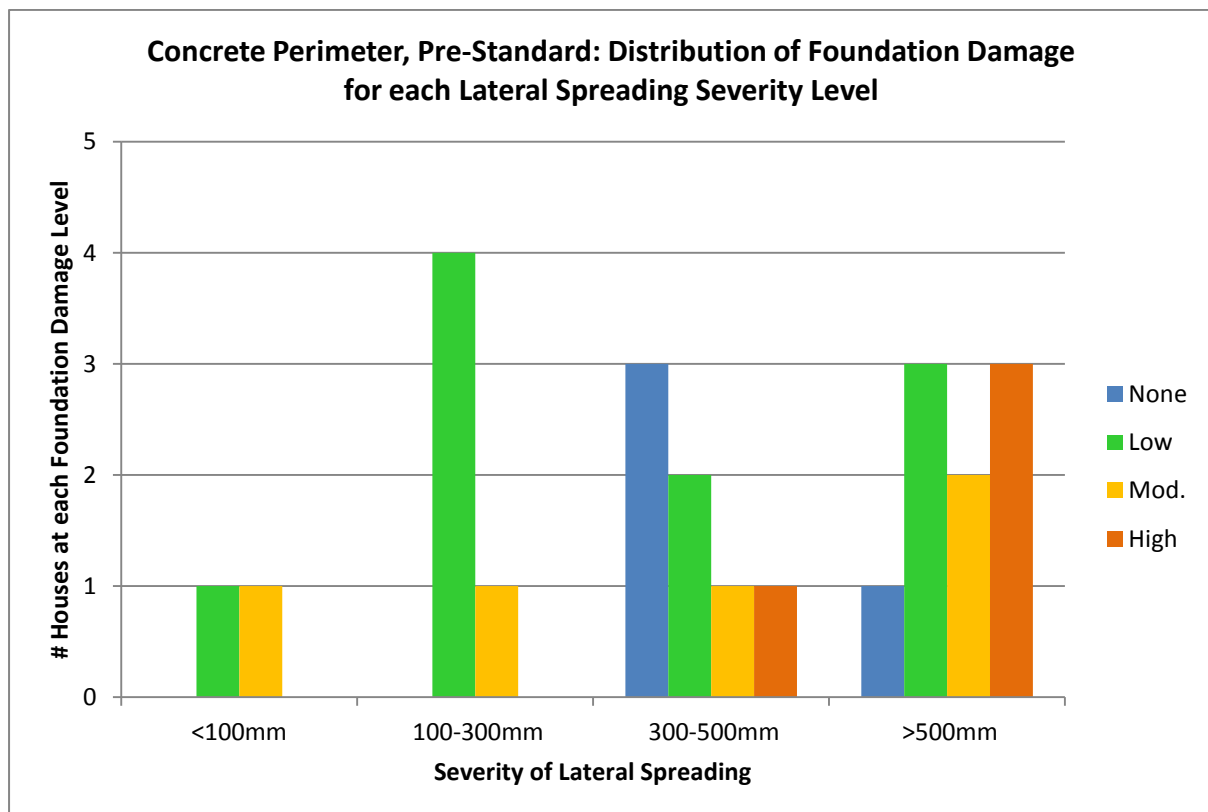


Figure A. 48. Distribution of foundation damage level for differing degrees of lateral ground movement for pre-standard, concrete perimeter foundations.

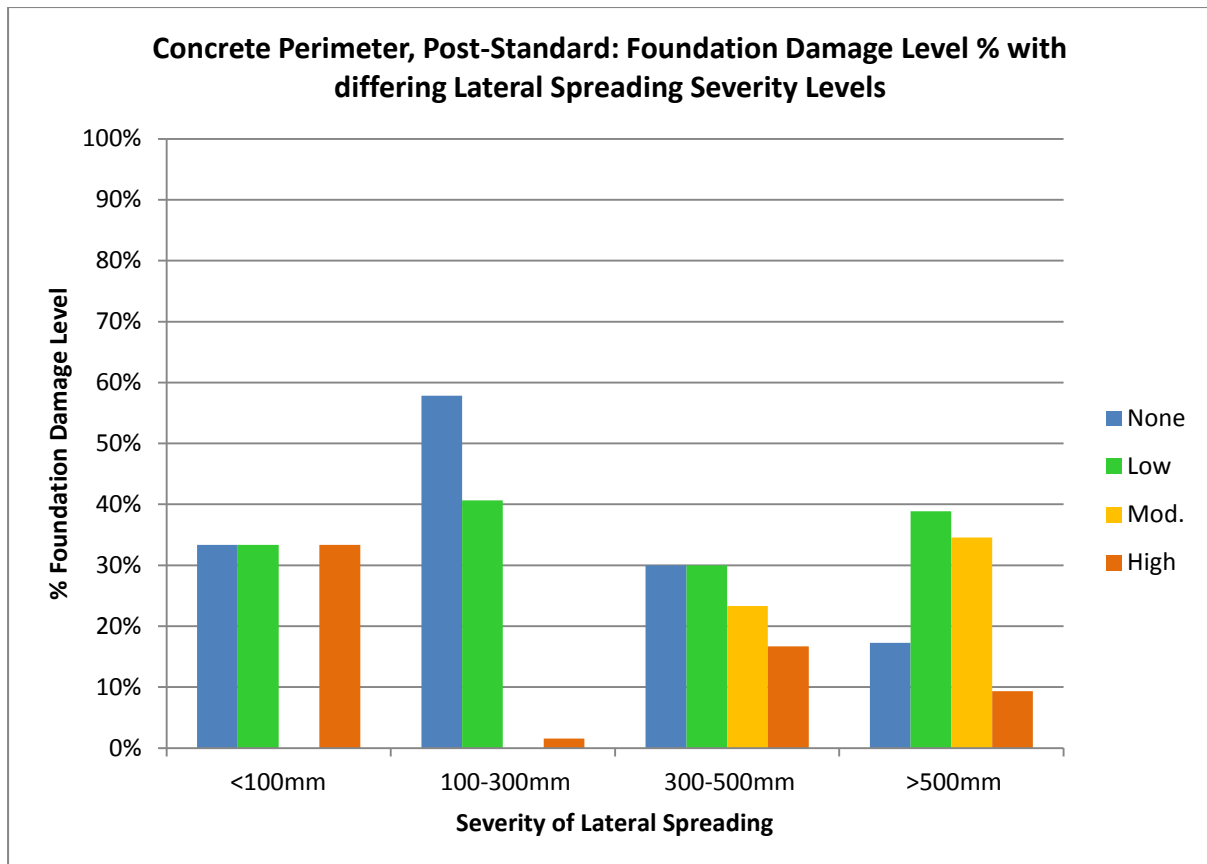


Figure A. 49. Distribution of foundation damage level in percentage for differing degrees of lateral ground movement for post-standard, concrete perimeter foundations.

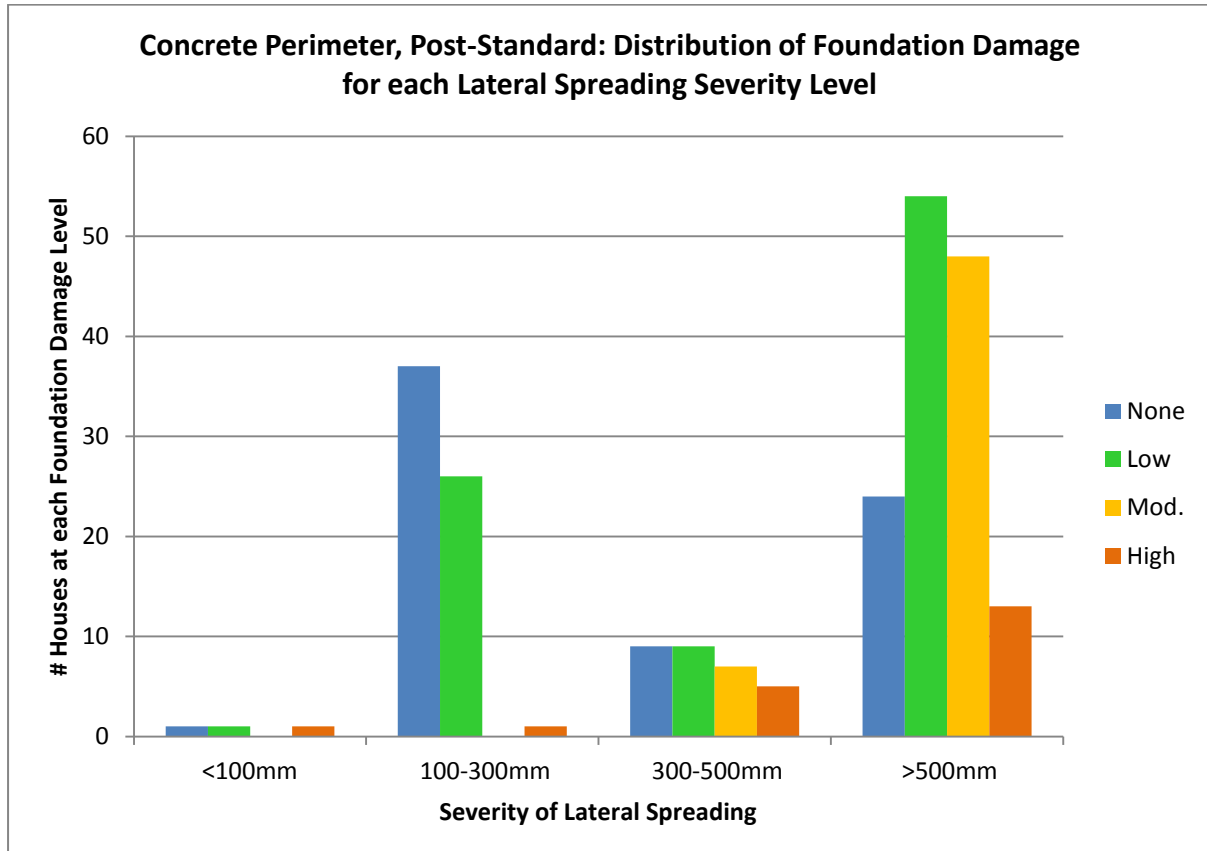


Figure A. 50. Distribution of foundation damage level for differing degrees of lateral ground movement for post-standard, concrete perimeter foundations.

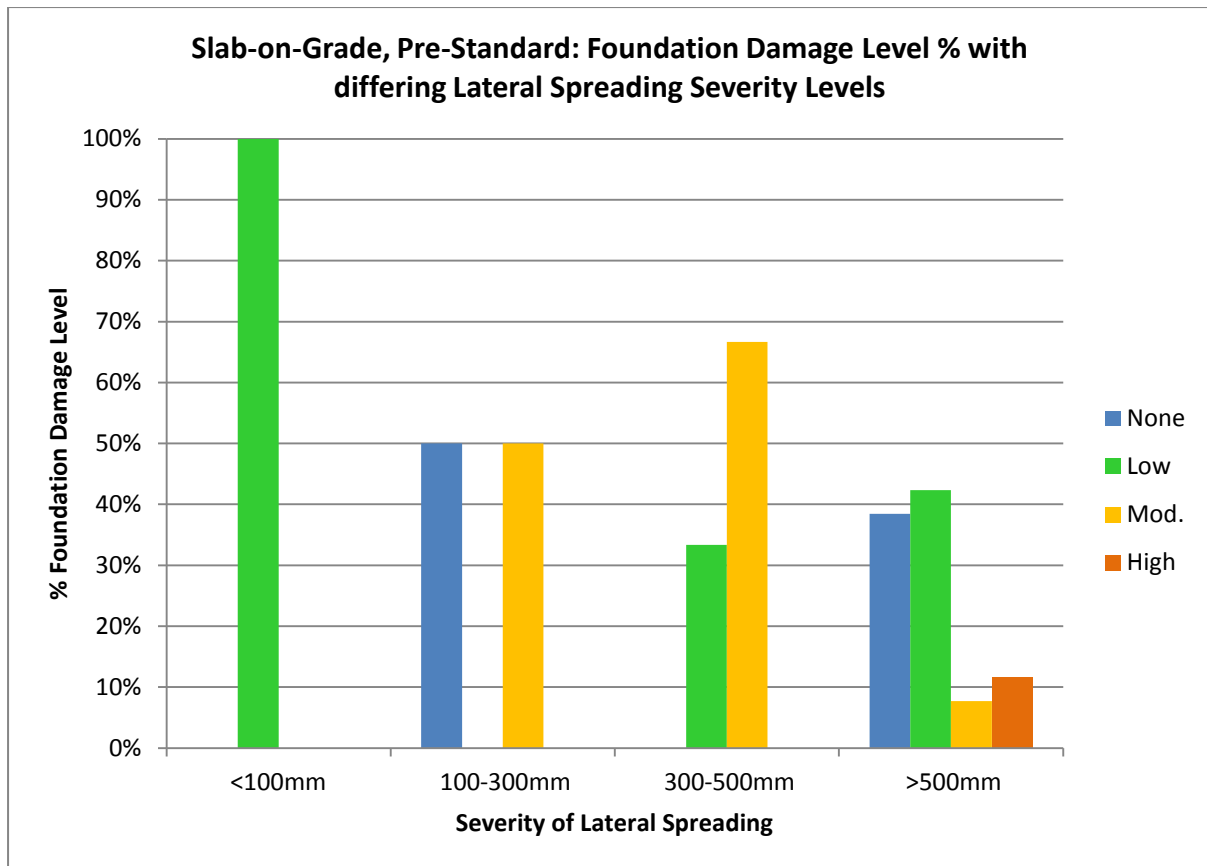


Figure A. 51. Distribution of foundation damage level in percentage for differing degrees of lateral ground movement for pre-standard, slab-on-grade foundations.

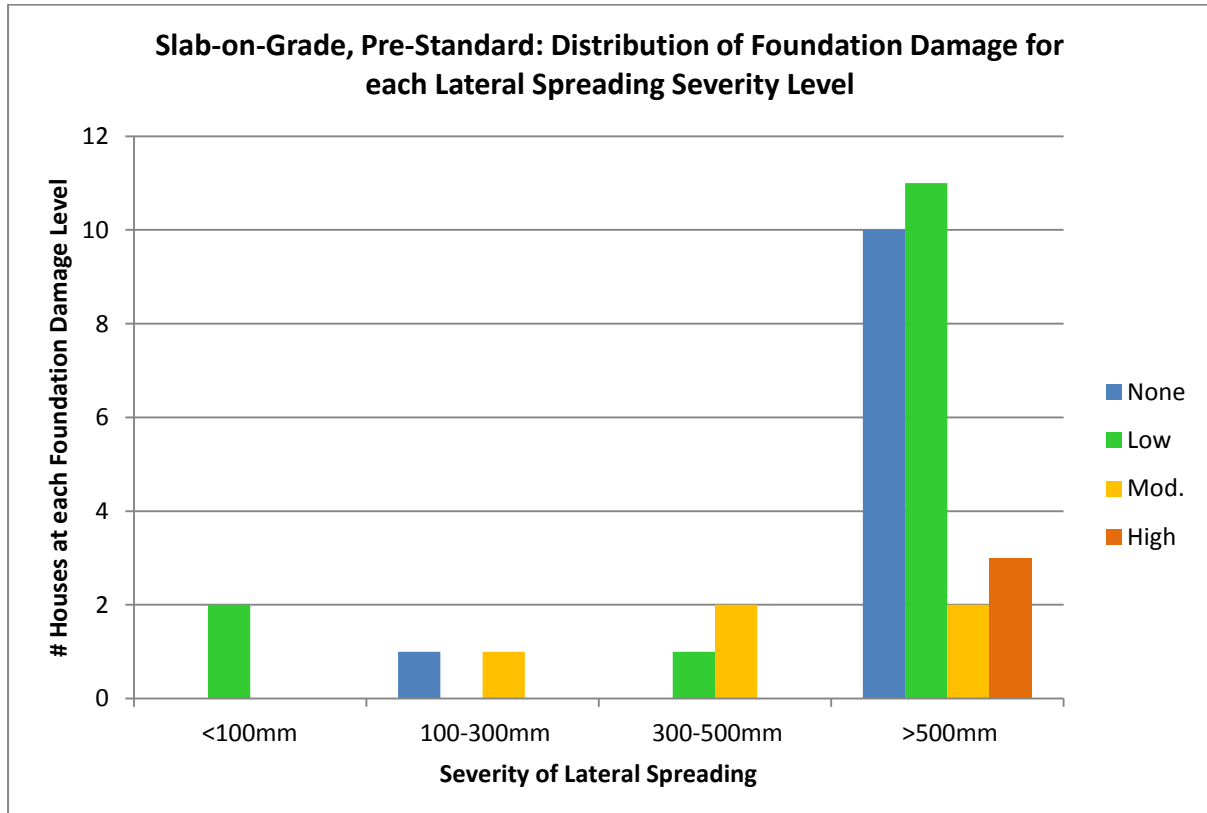


Figure A. 52. Distribution of foundation damage level for differing degrees of lateral ground movement for pre-standard, slab-on-grade foundations.

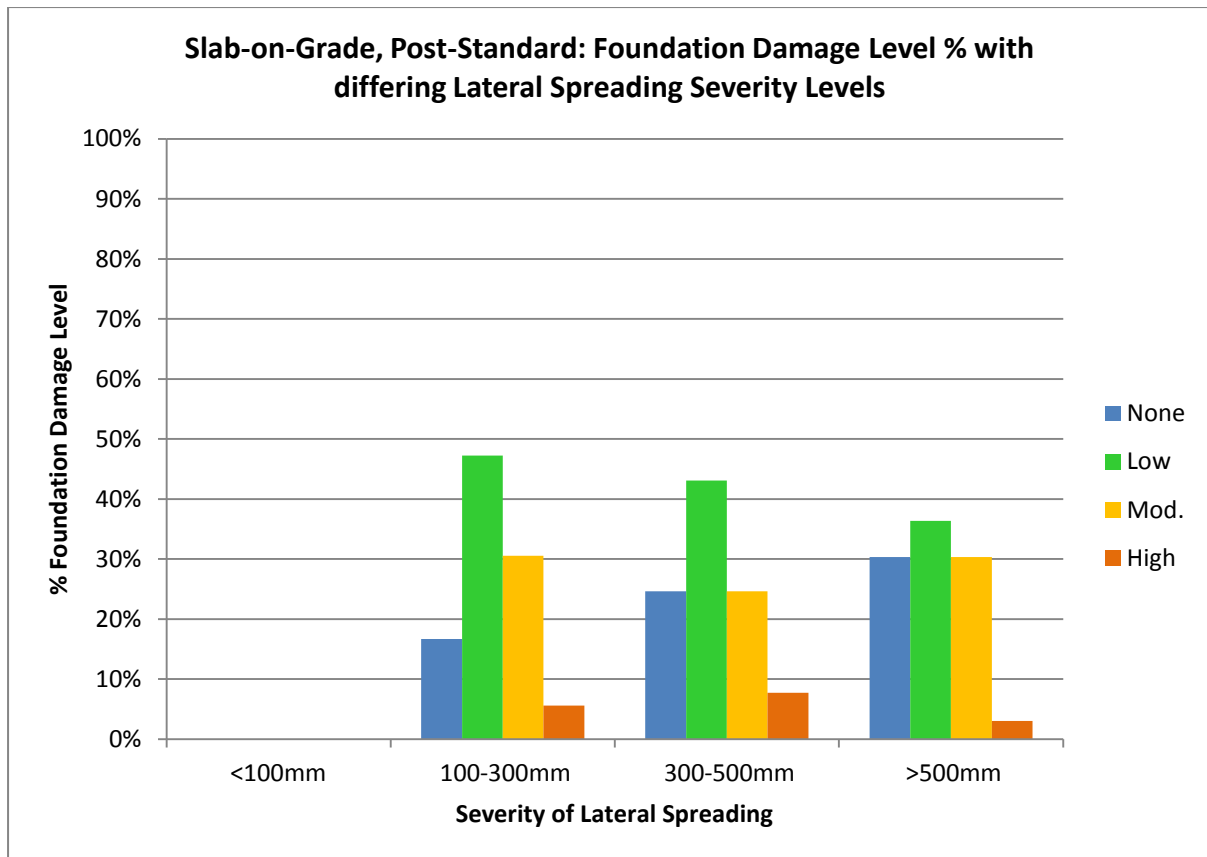


Figure A. 53. Distribution of foundation damage level in percentage for differing degrees of lateral ground movement for post-standard, slab-on-grade foundations.

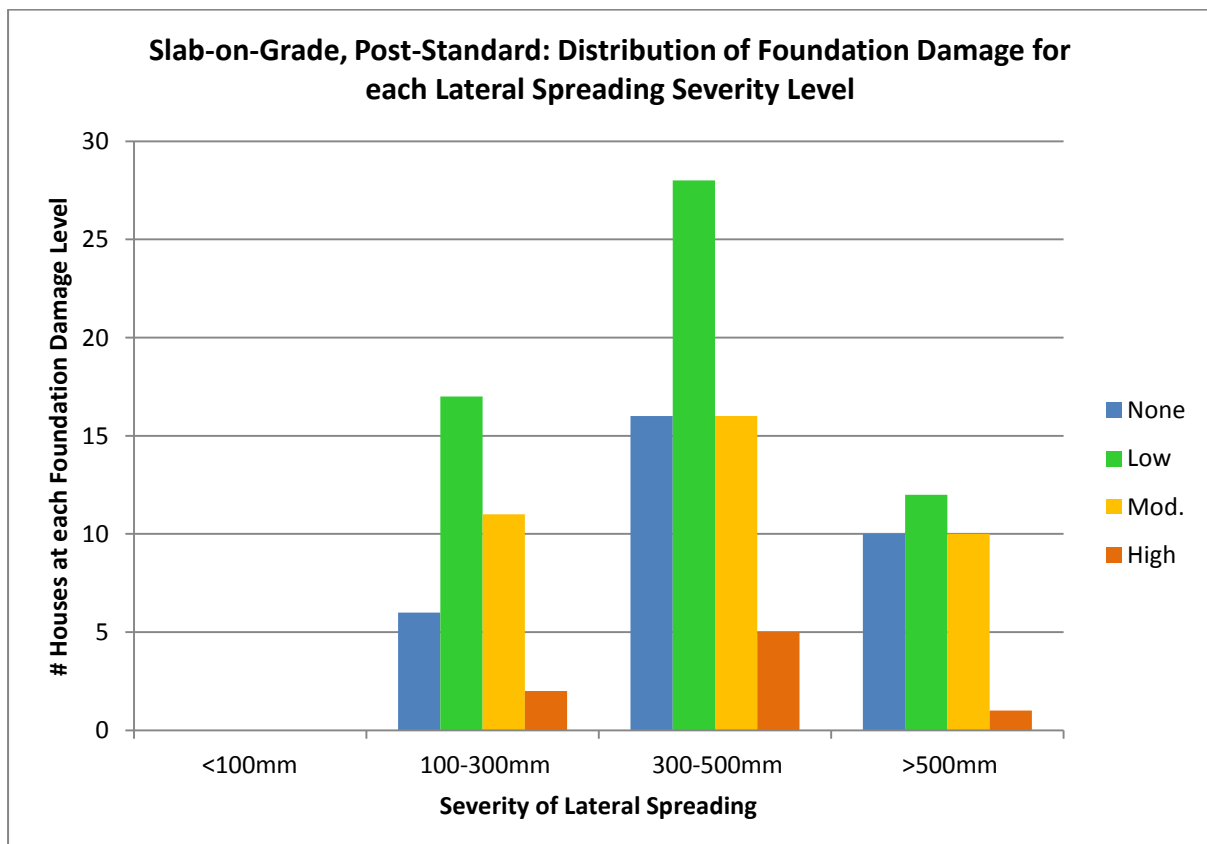


Figure A. 54. Distribution of foundation damage level for differing degrees of lateral ground movement for post-standard, slab-on-grade foundations.

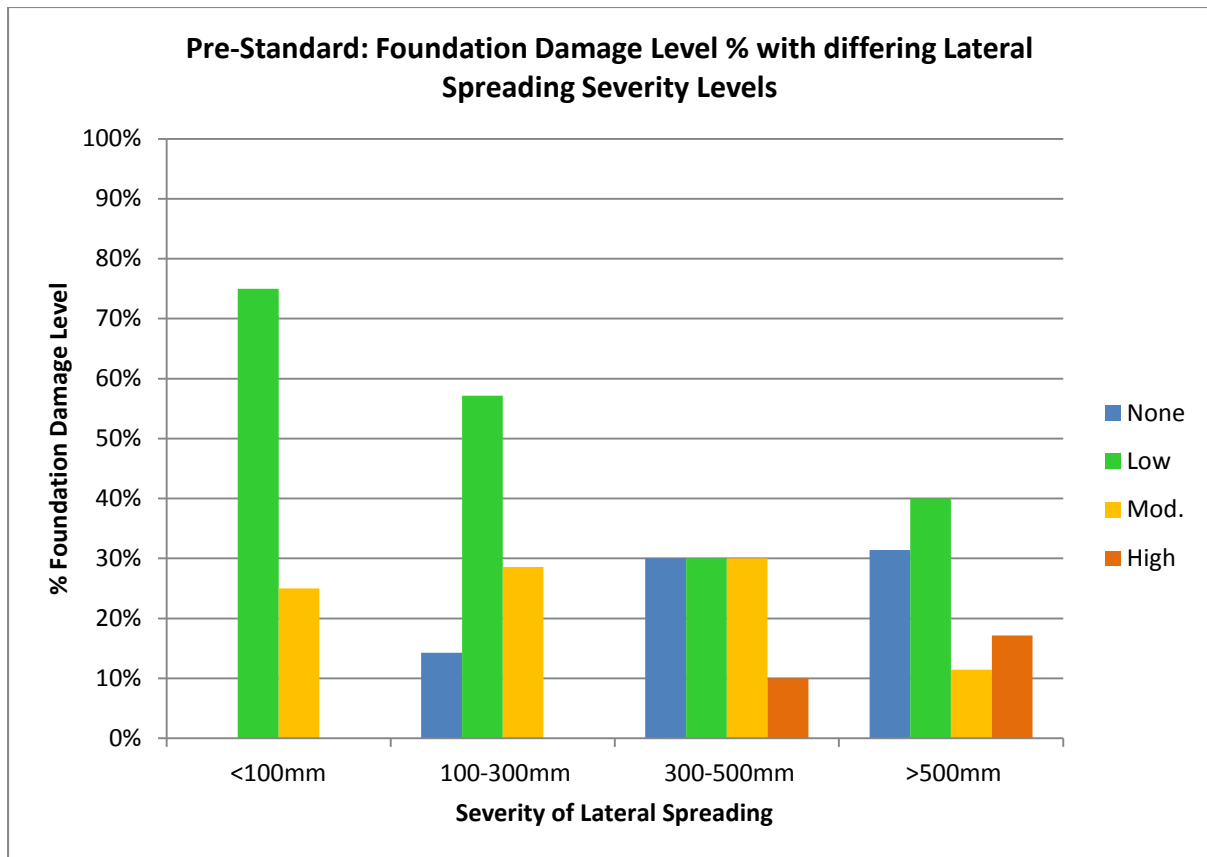


Figure A. 55. Distribution of foundation damage level in percentage for differing degrees of lateral ground movement for all pre-standard foundations.

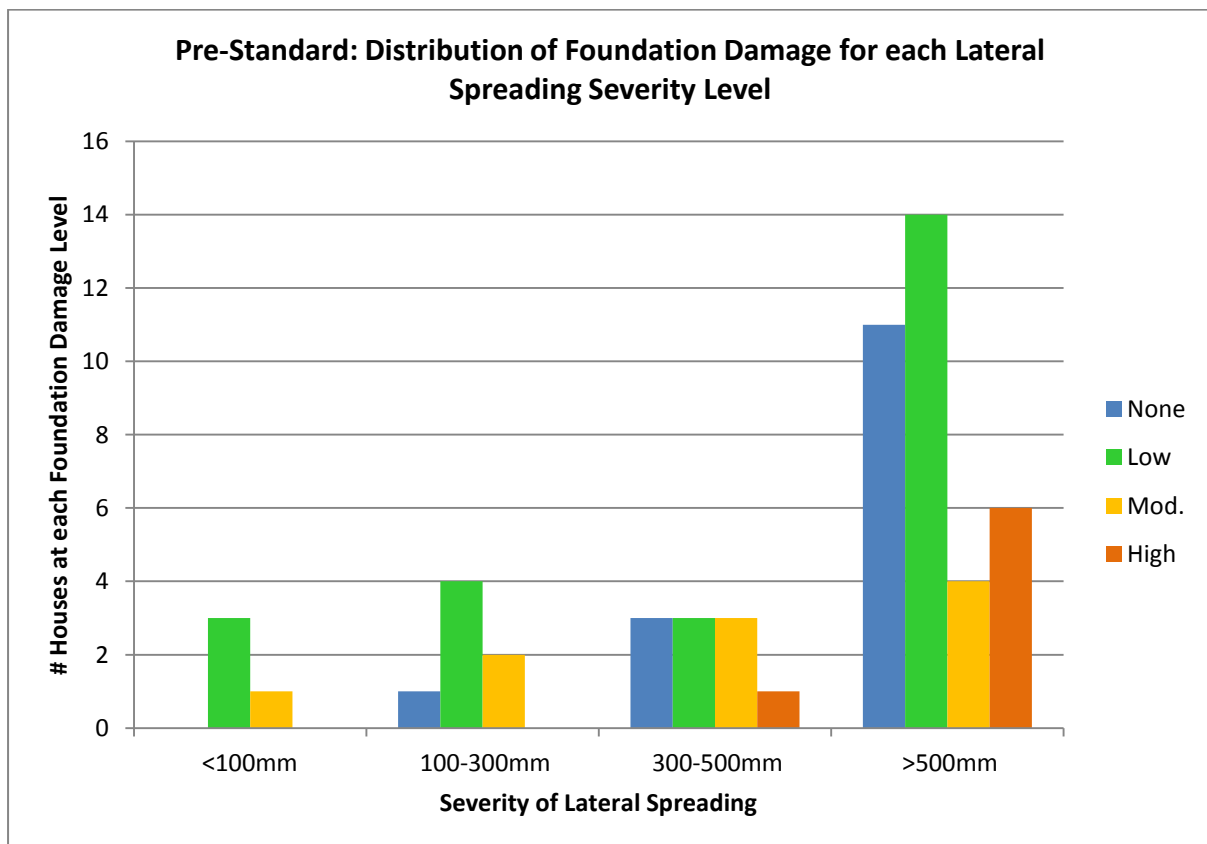


Figure A. 56. Distribution of foundation damage level for differing degrees of lateral ground movement for all pre-standard foundations.

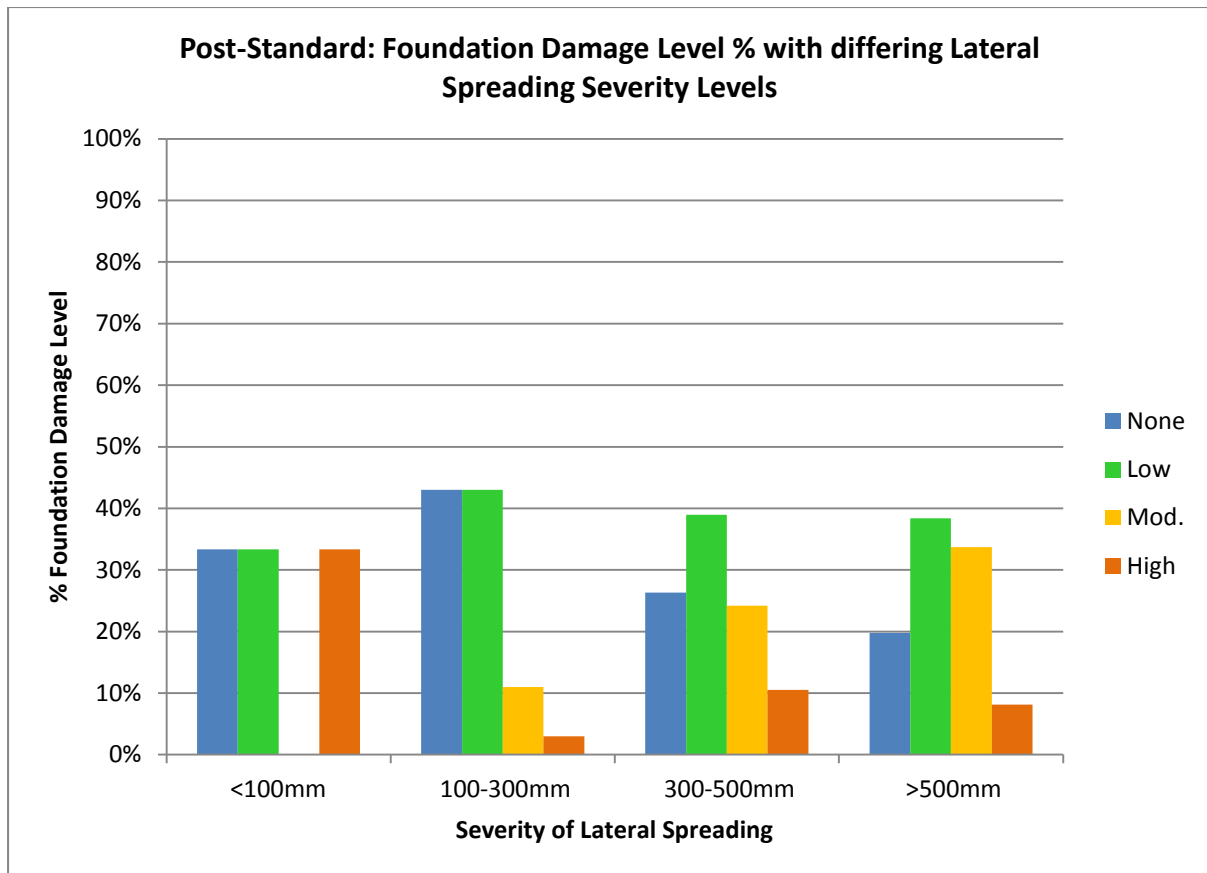


Figure A. 57. Distribution of foundation damage level in percentage for differing degrees of lateral ground movement for all post-standard foundations.

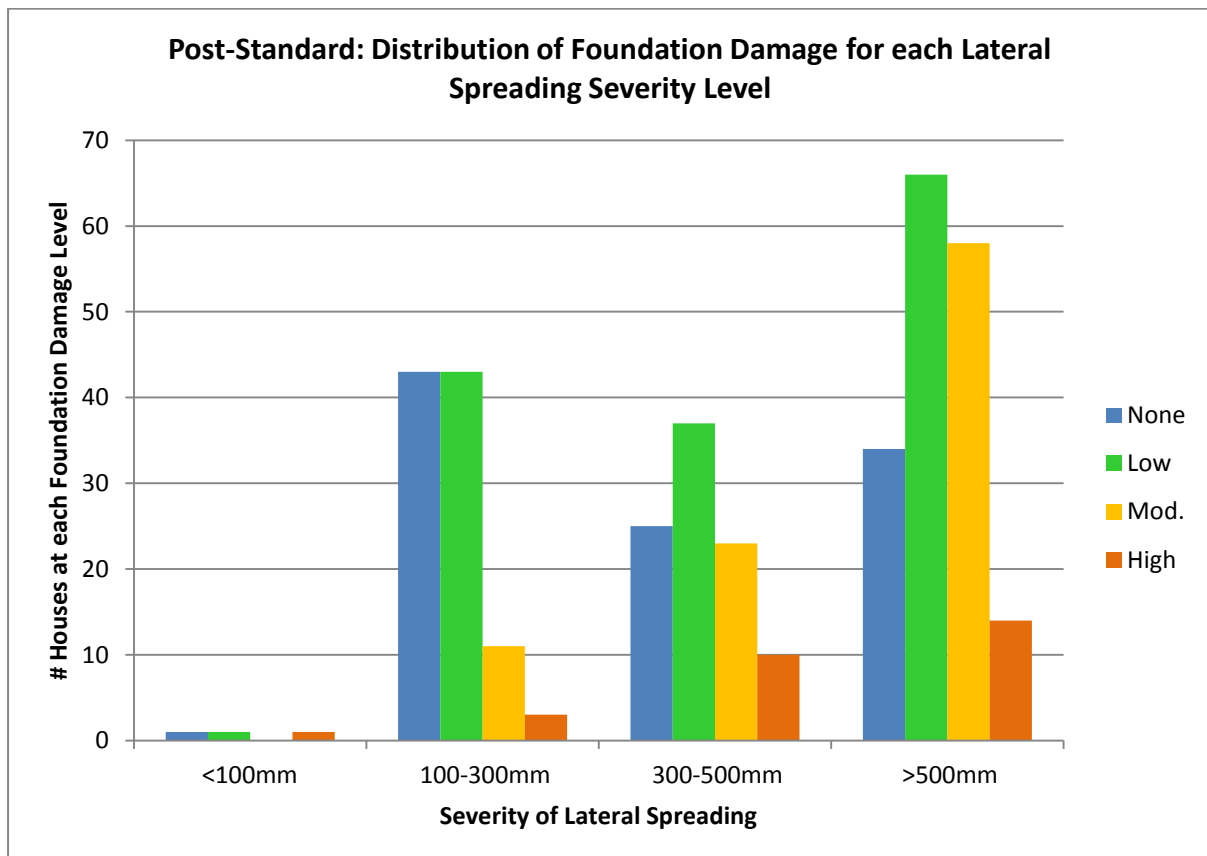


Figure A. 58. Distribution of foundation damage level in percentage for differing degrees of lateral ground movement for all post-standard foundations.

E.3.3: By Building Weight

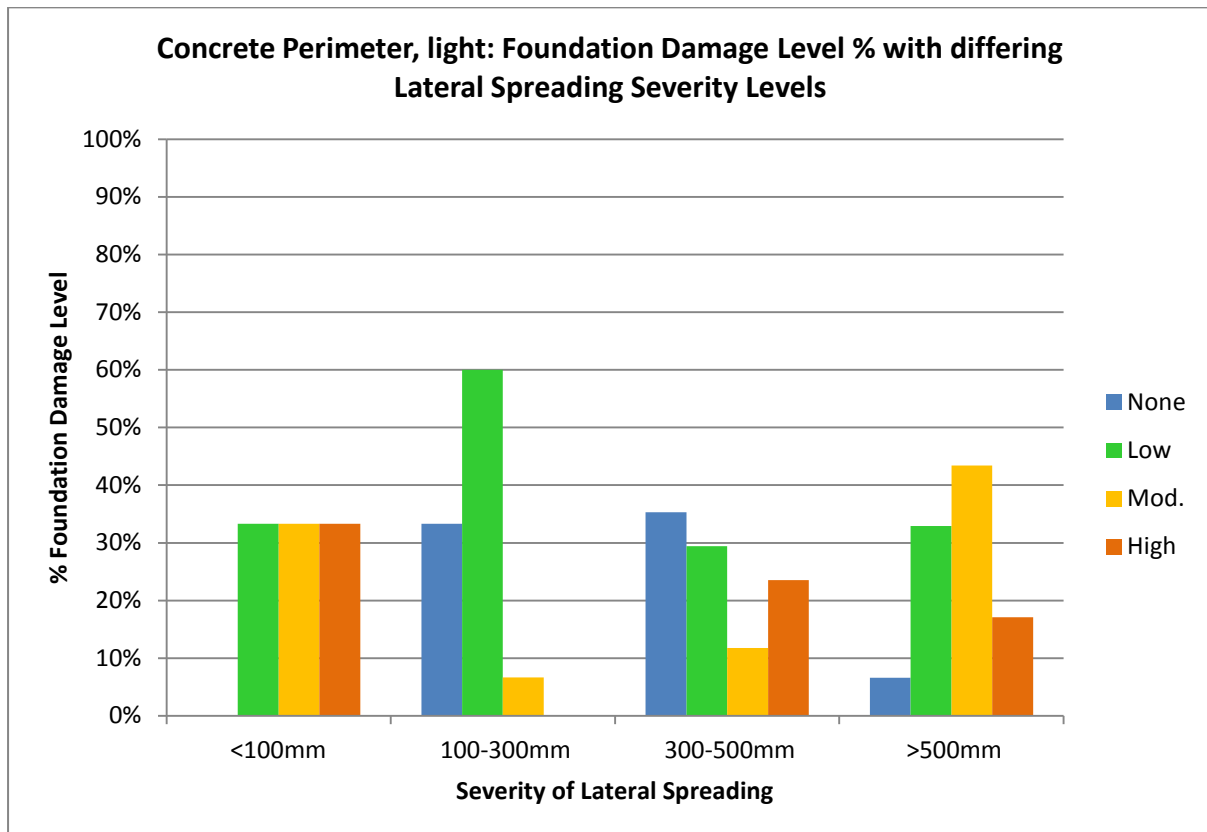


Figure A. 59. Distribution of foundation damage level in percentage for differing degrees of lateral ground movement for light buildings on concrete perimeter foundations.

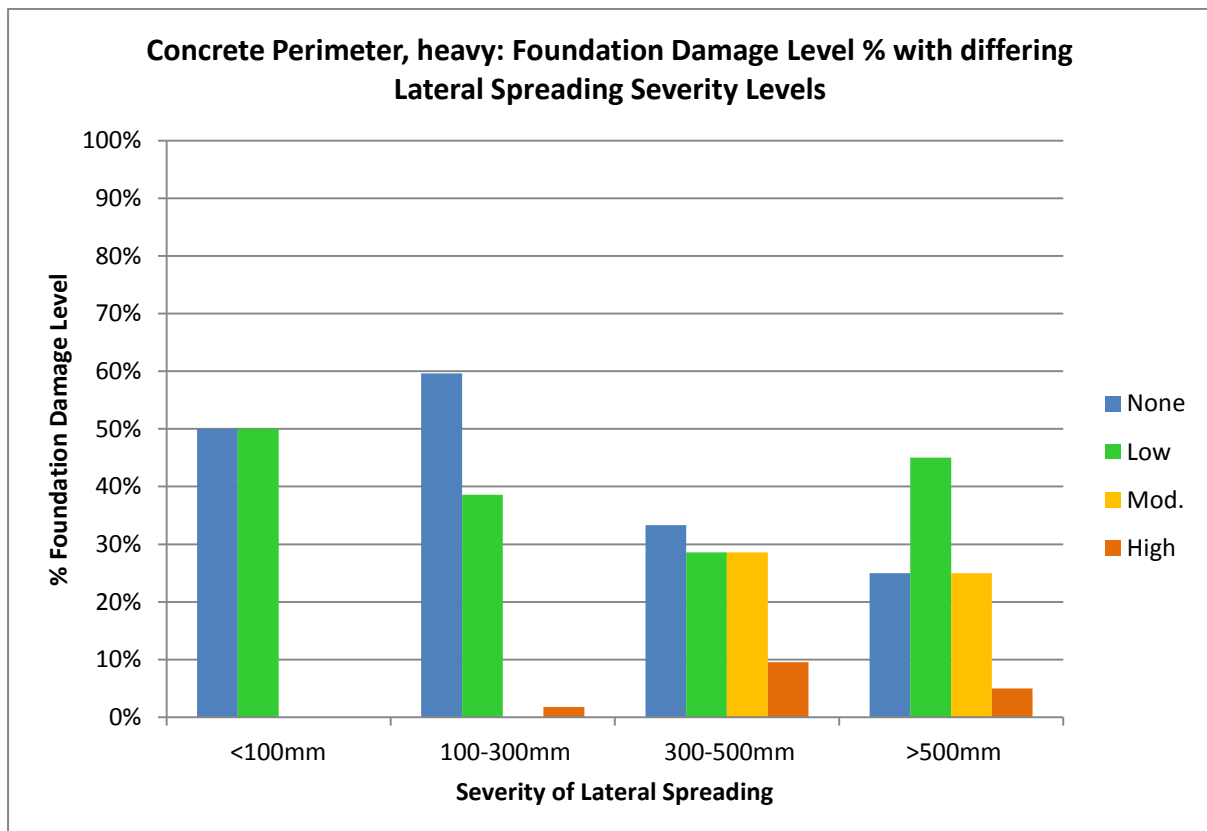


Figure A. 60. Distribution of foundation damage level in percentage for differing degrees of lateral ground movement for heavy buildings on concrete perimeter foundations.

E.4: Foundation Damage vs. Superstructure Damage

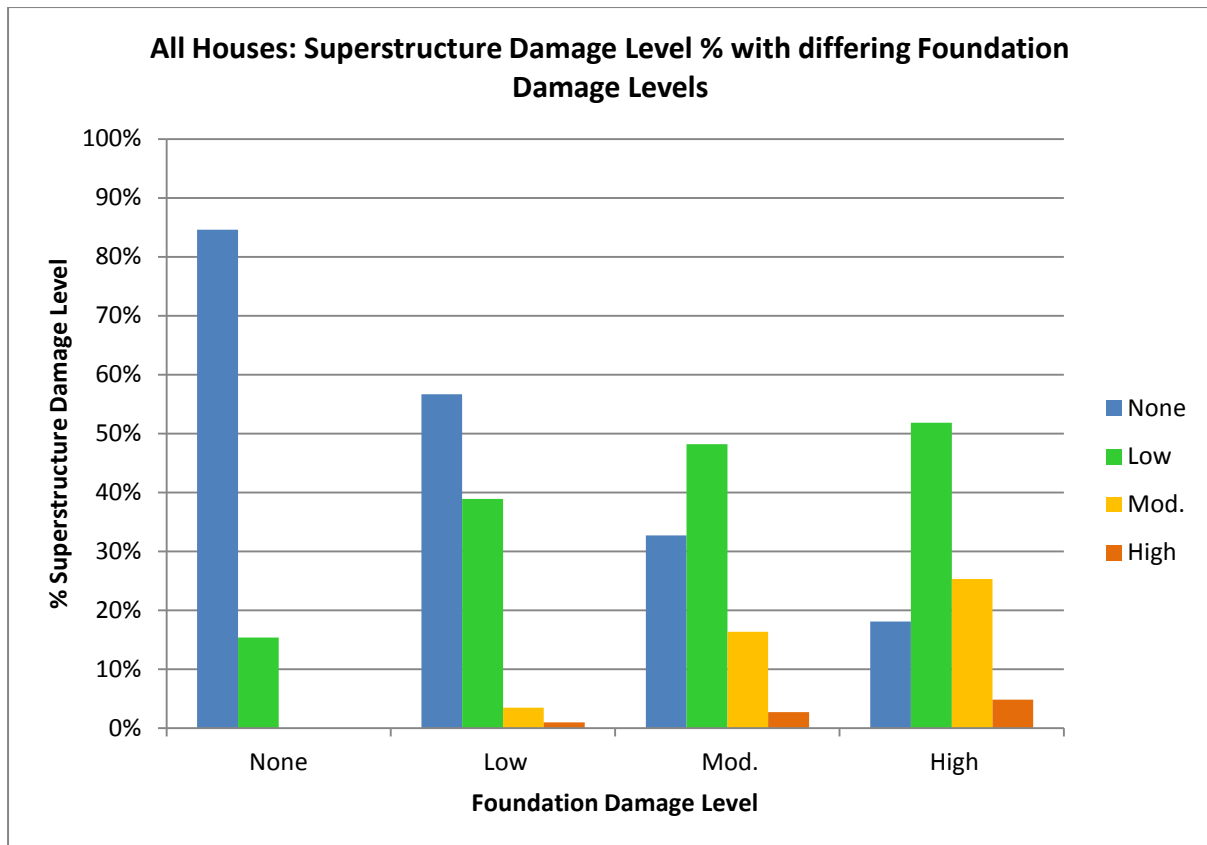


Figure A. 61. Distribution in percentage of damage to house superstructures for differing foundation damage levels for all houses.

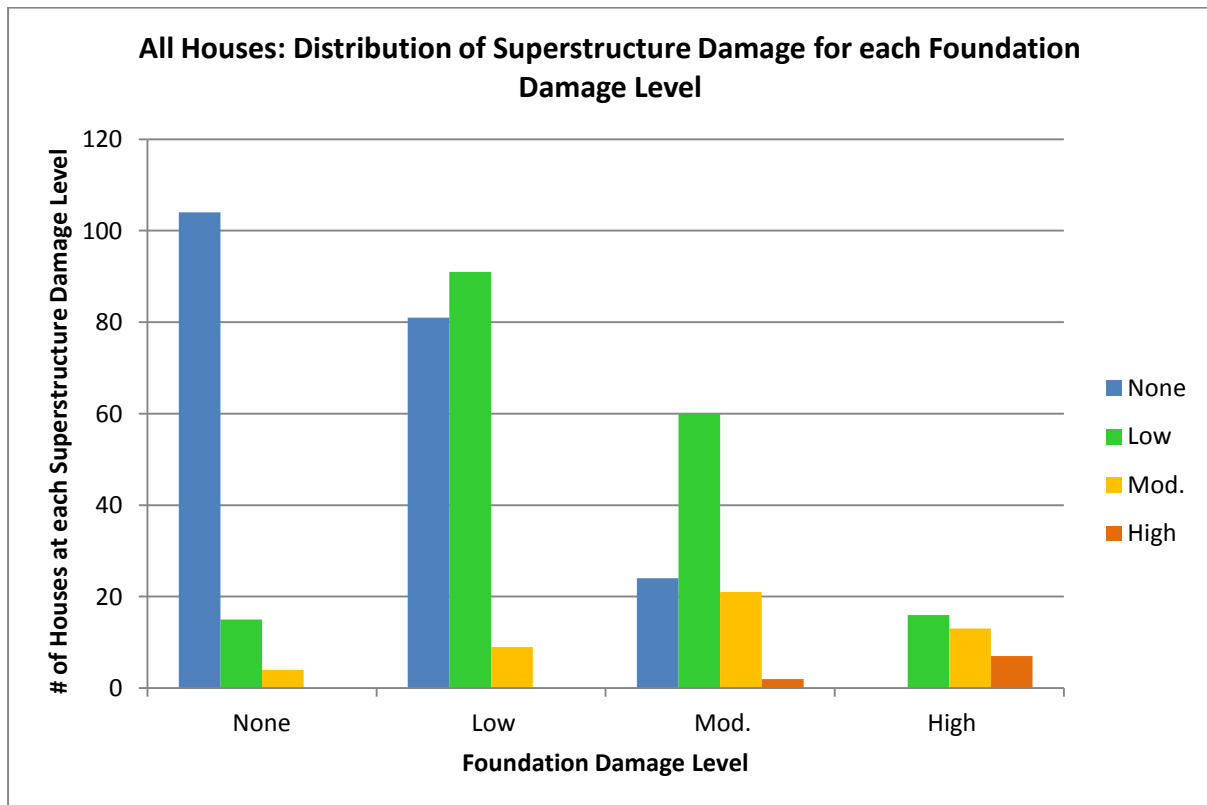


Figure A. 62. Distribution of damage to house superstructures for differing foundation damage levels for all houses.

E.4.1: By Foundation Type

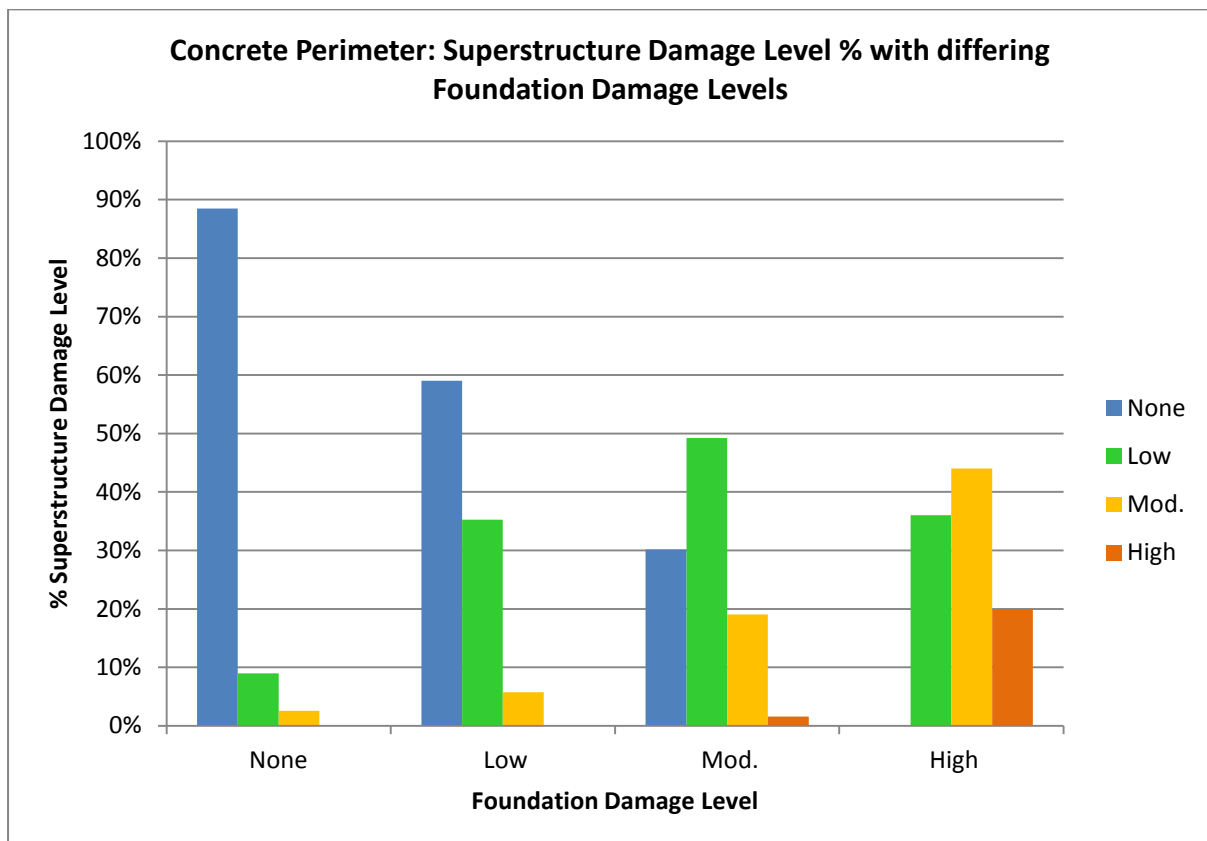


Figure A. 63. Distribution in percentage of damage to house superstructures for differing foundation damage levels for concrete perimeter foundations.

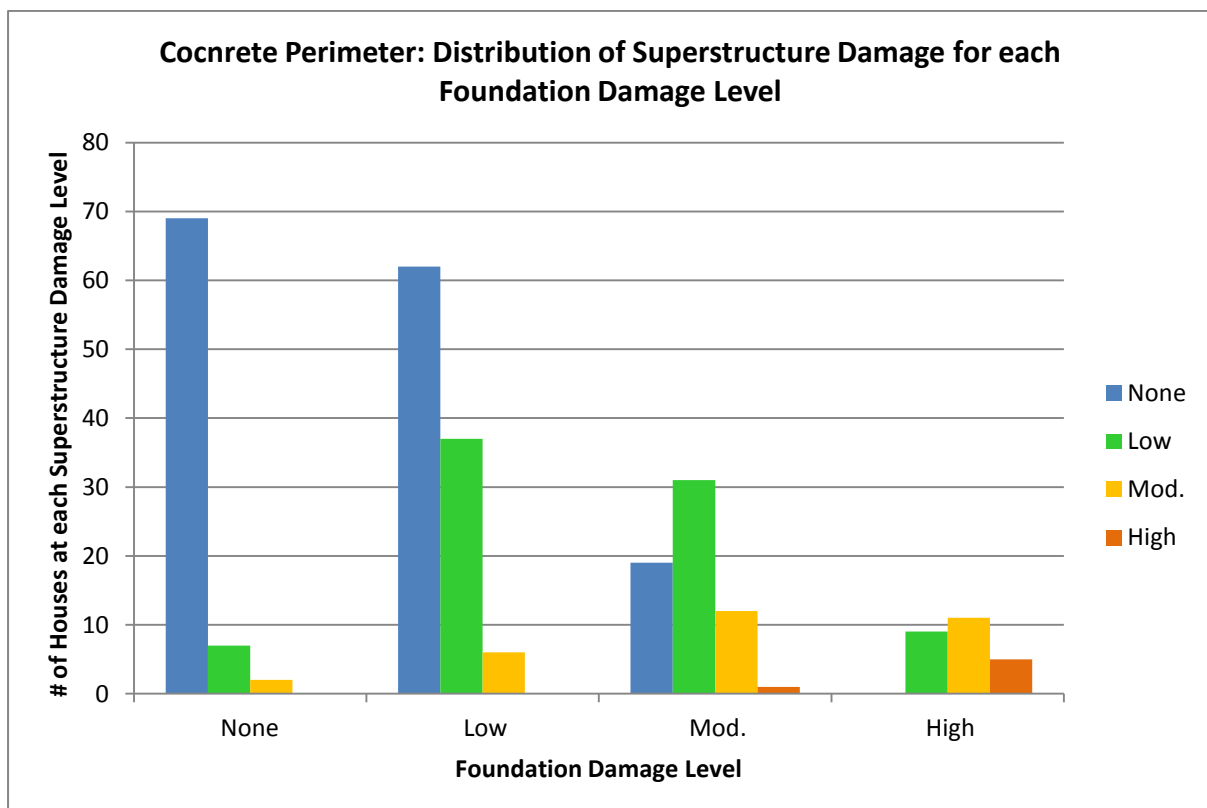


Figure A. 64. Distribution of damage to house superstructures for differing foundation damage levels for concrete perimeter foundations.

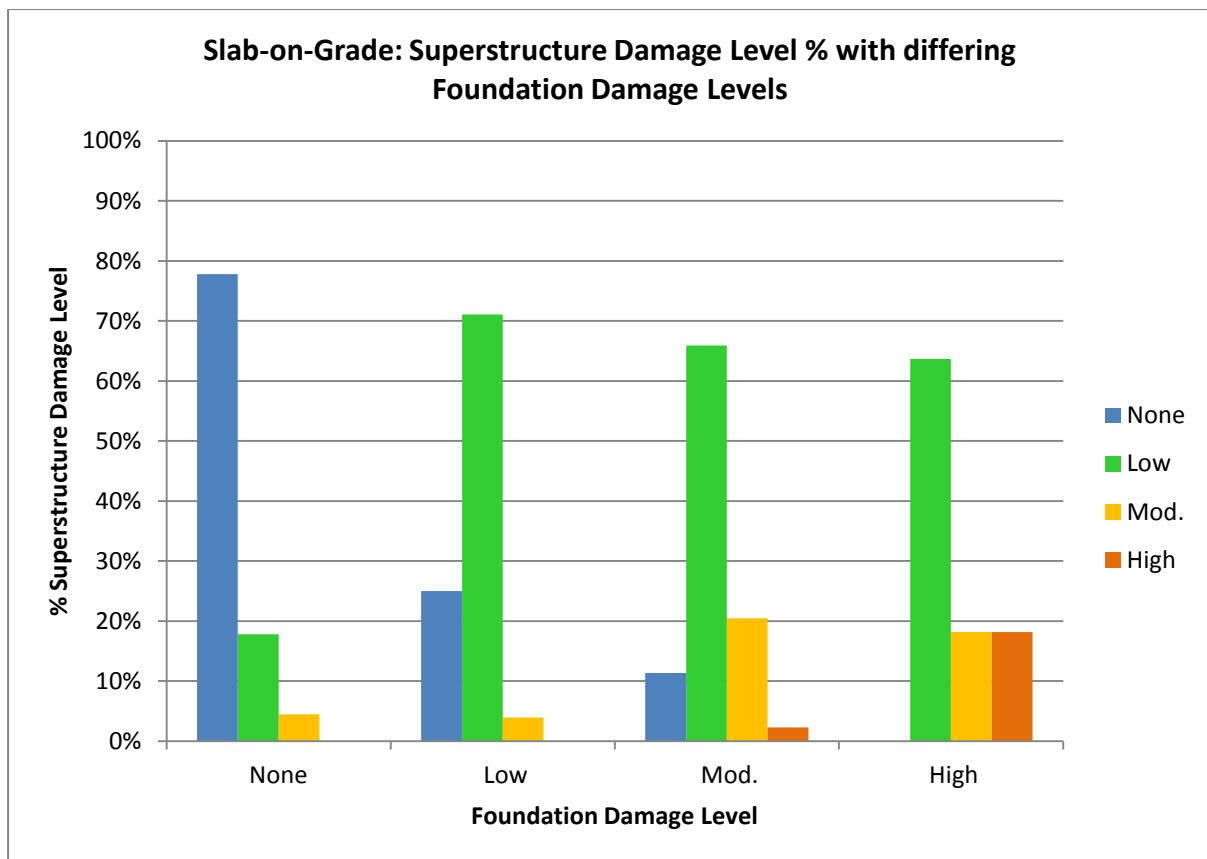


Figure A. 65. Distribution in percentage of damage to house superstructures for differing foundation damage levels for slab-on-grade foundations.

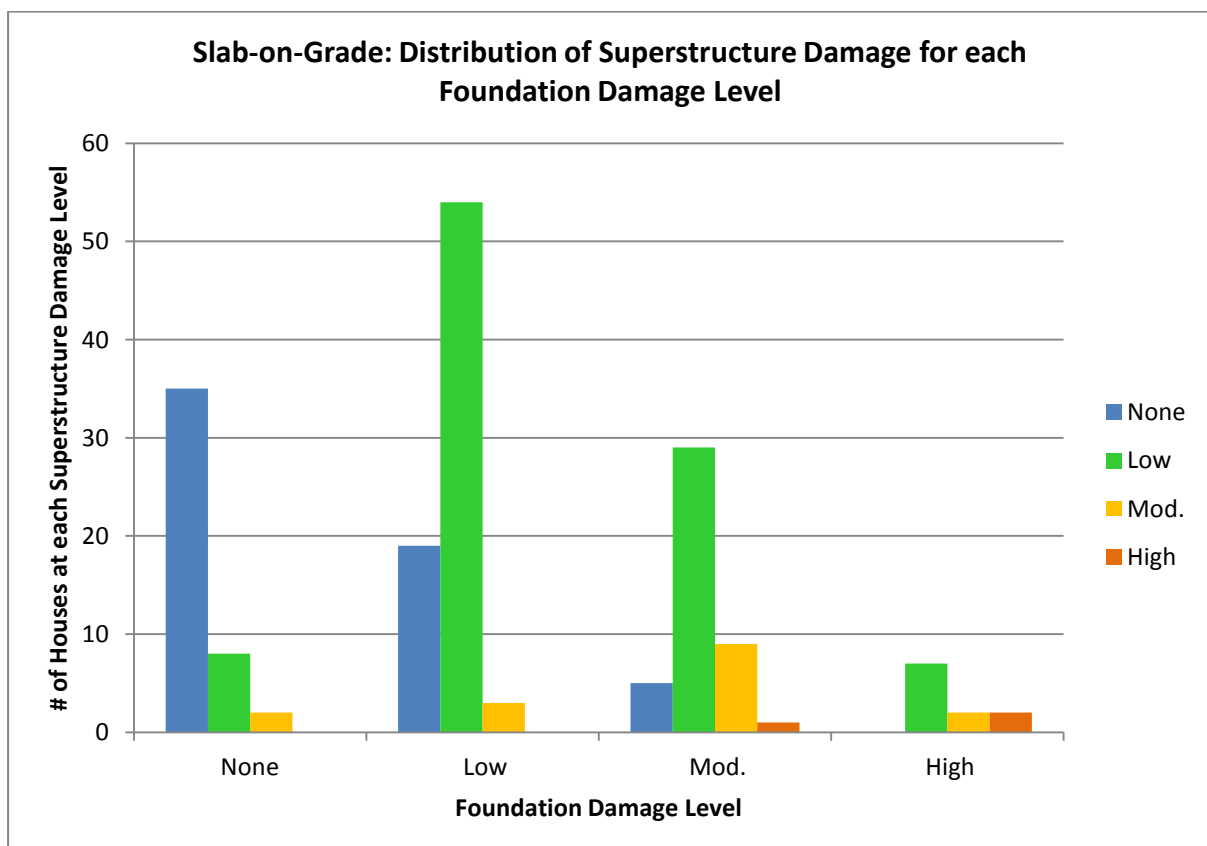


Figure A. 66. Distribution of damage to house superstructures for differing foundation damage levels for slab-on-grade foundations.

E.4.2: By Age

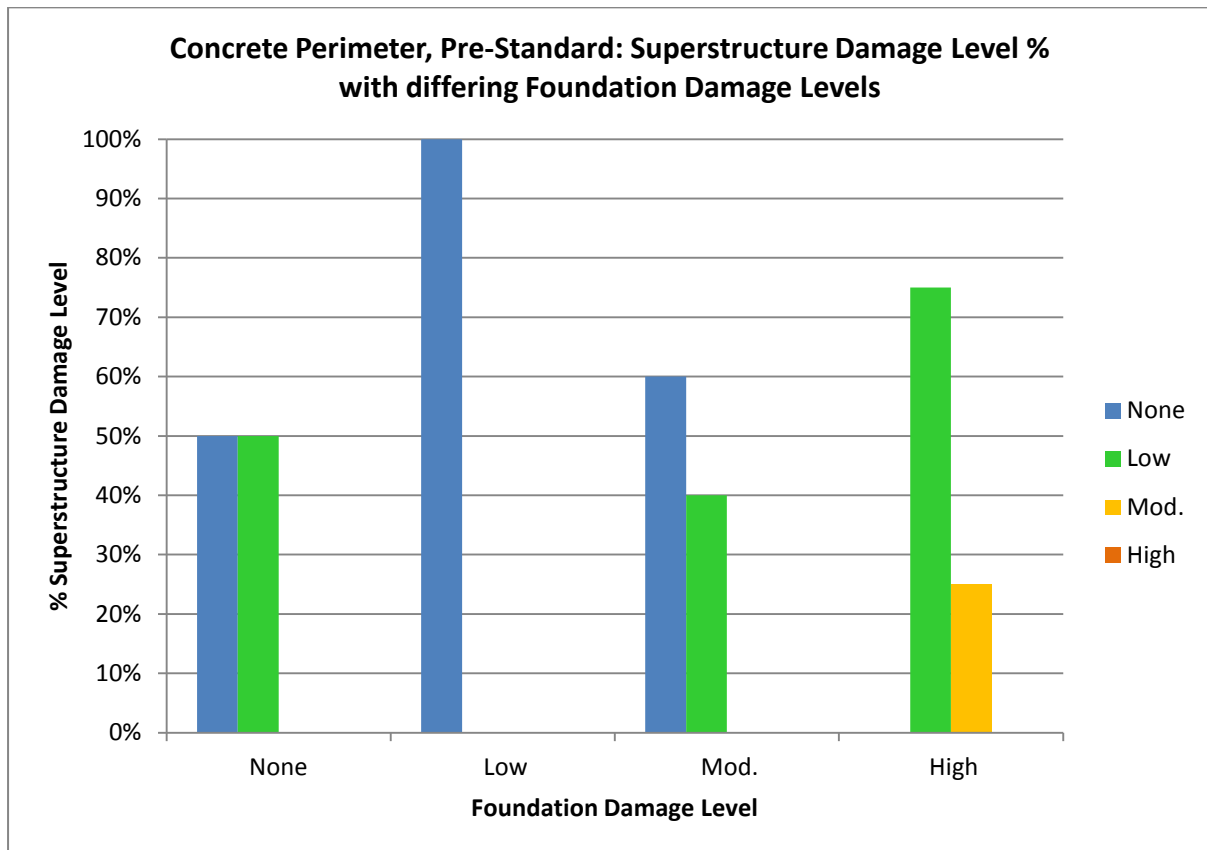


Figure A. 67. Distribution in percentage of damage to house superstructures for differing foundation damage levels for pre-standard, concrete perimeter foundations.

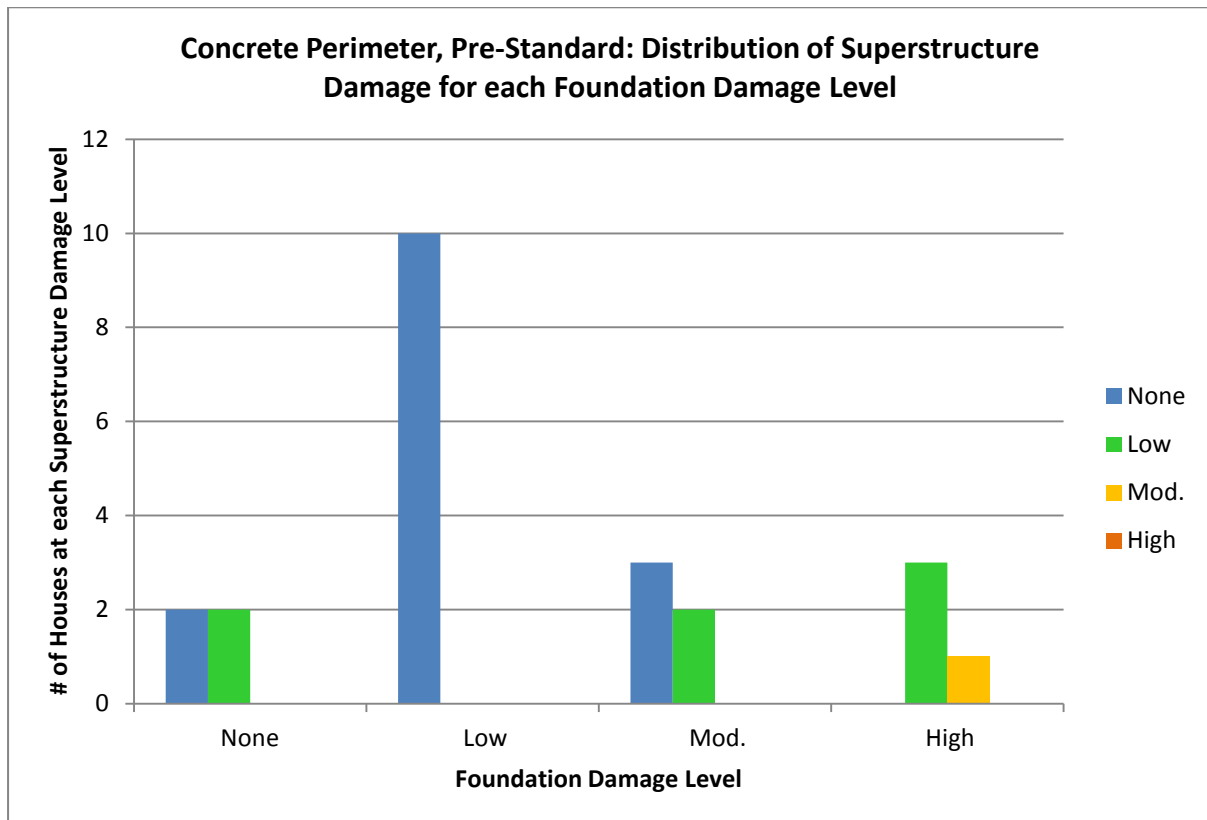


Figure A. 68. Distribution of damage to house superstructures for differing foundation damage levels for pre-standard, concrete perimeter foundations.

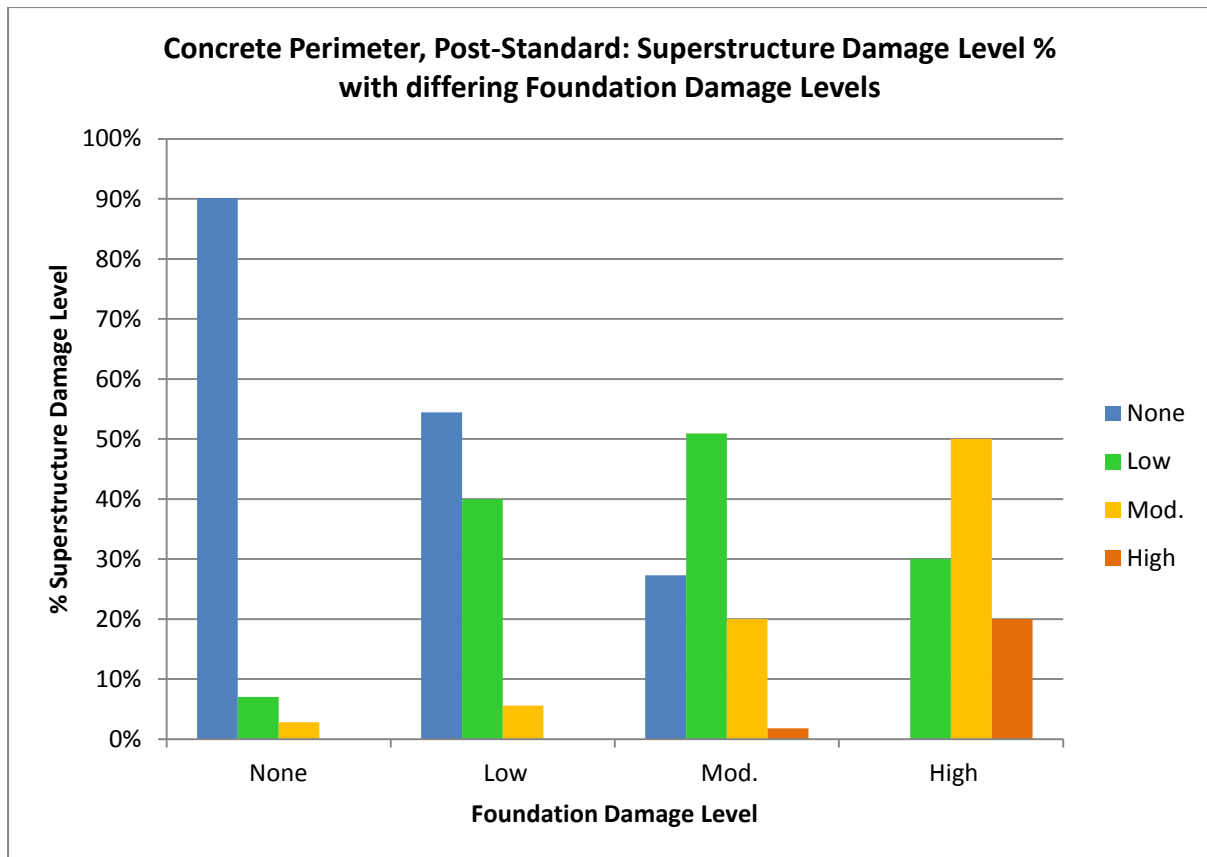


Figure A. 69. Distribution in percentage of damage to house superstructures for differing foundation damage levels for post-standard, concrete perimeter foundations.

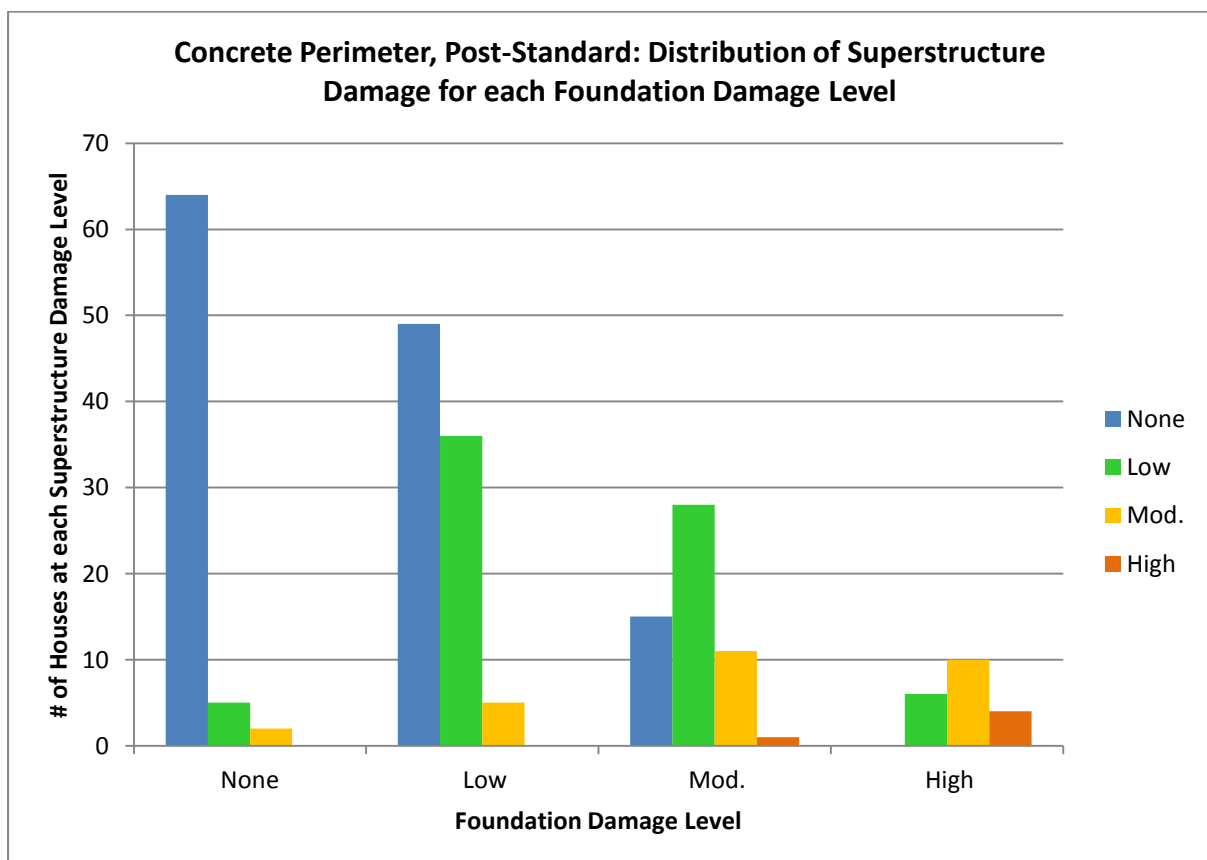


Figure A. 70. Distribution of damage to house superstructures for differing foundation damage levels for post-standard, concrete perimeter foundations.

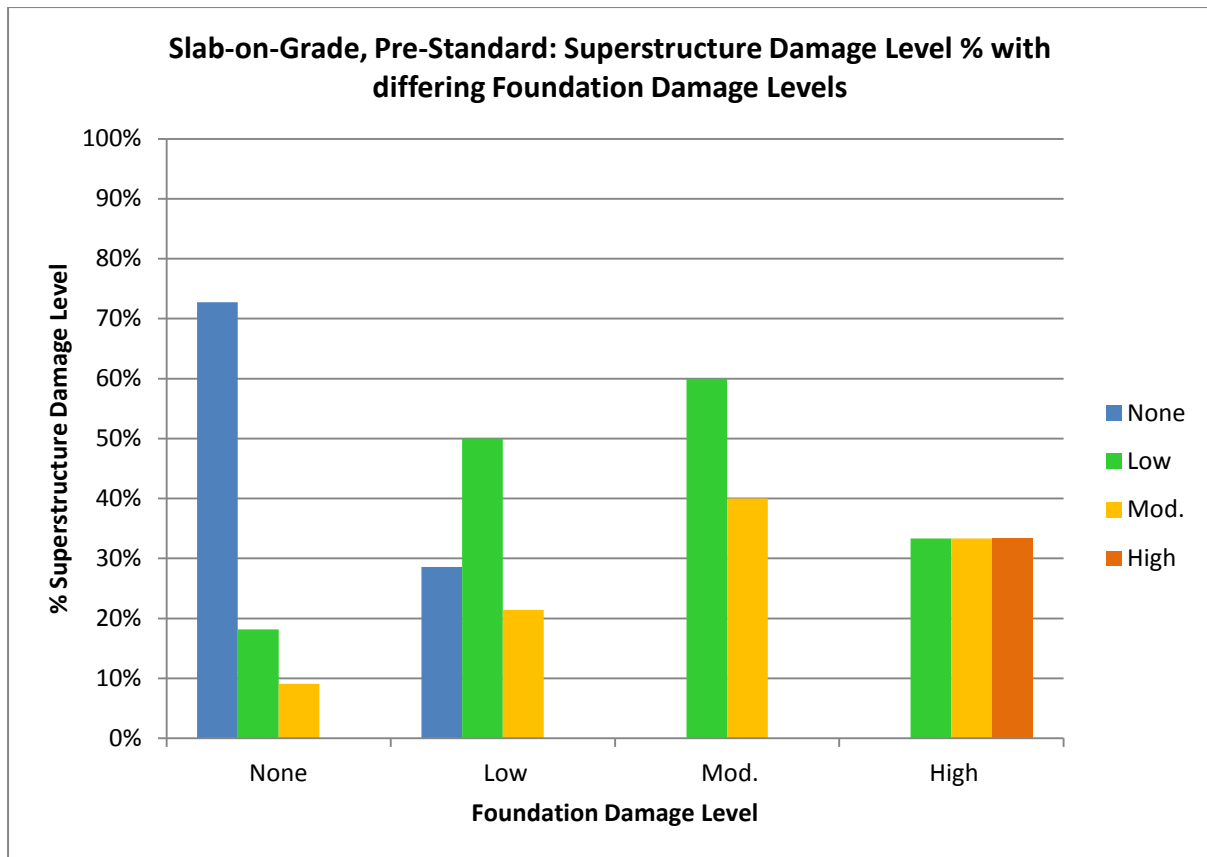


Figure A. 71. Distribution in percentage of damage to house superstructures for differing foundation damage levels for pre-standard, slab-on-grade foundations.

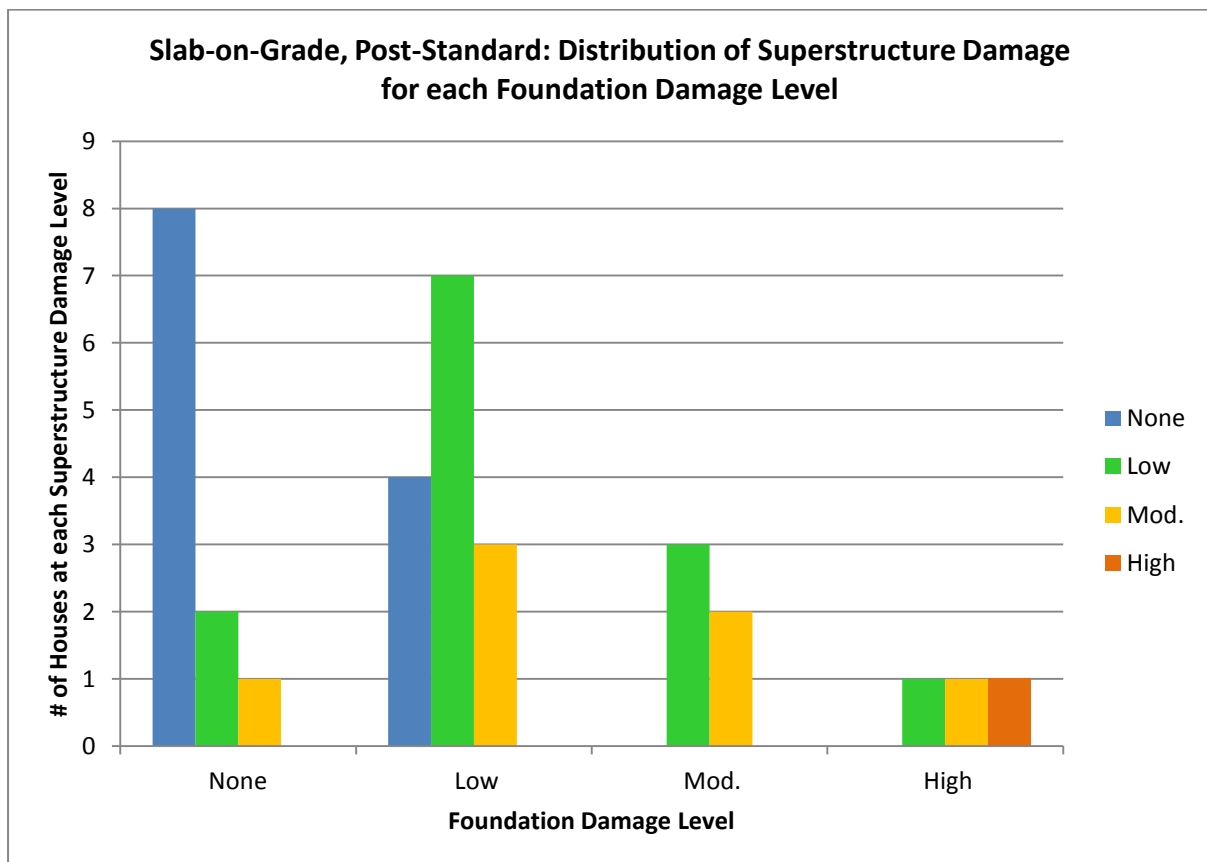


Figure A. 72. Distribution of damage to house superstructures for differing foundation damage levels for pre-standard, slab-on-grade foundations.

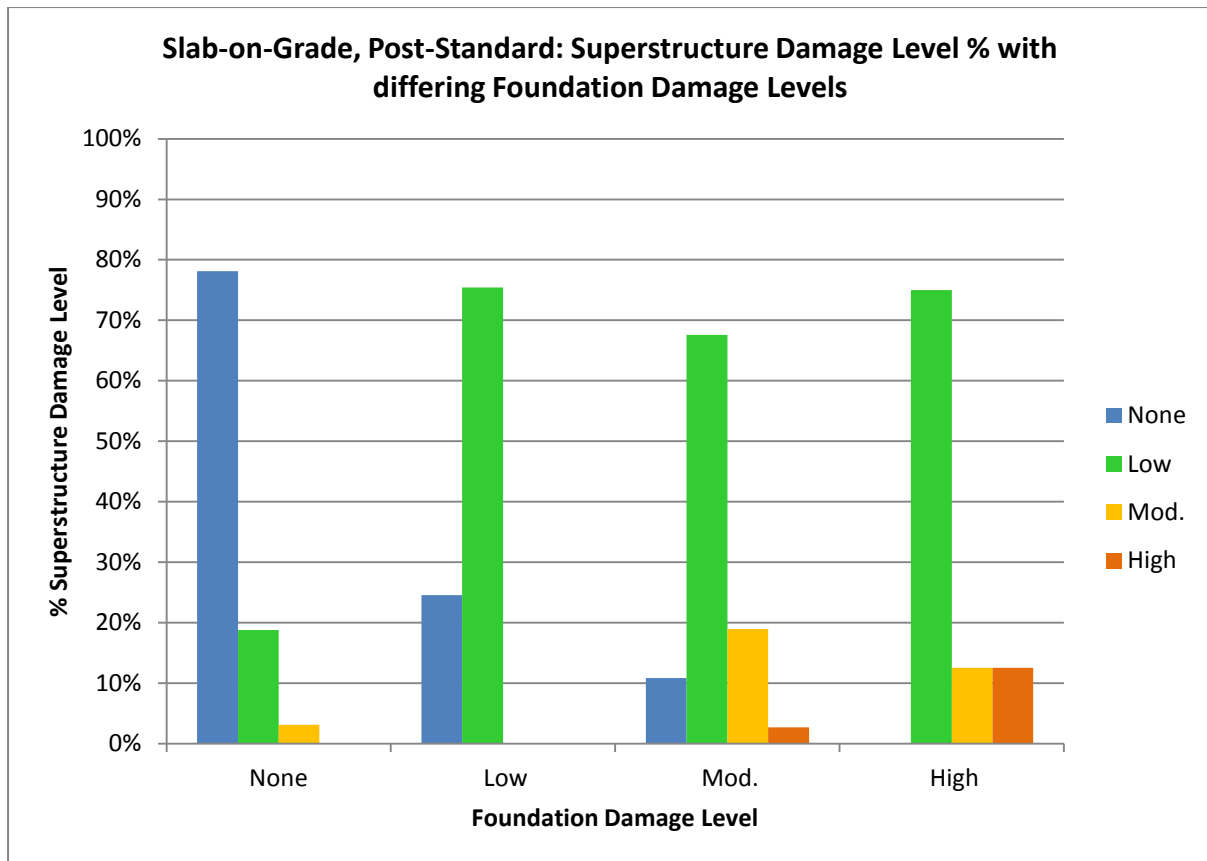


Figure A. 73. Distribution in percentage of damage to house superstructures for differing foundation damage levels for post-standard, slab-on-grade foundations.

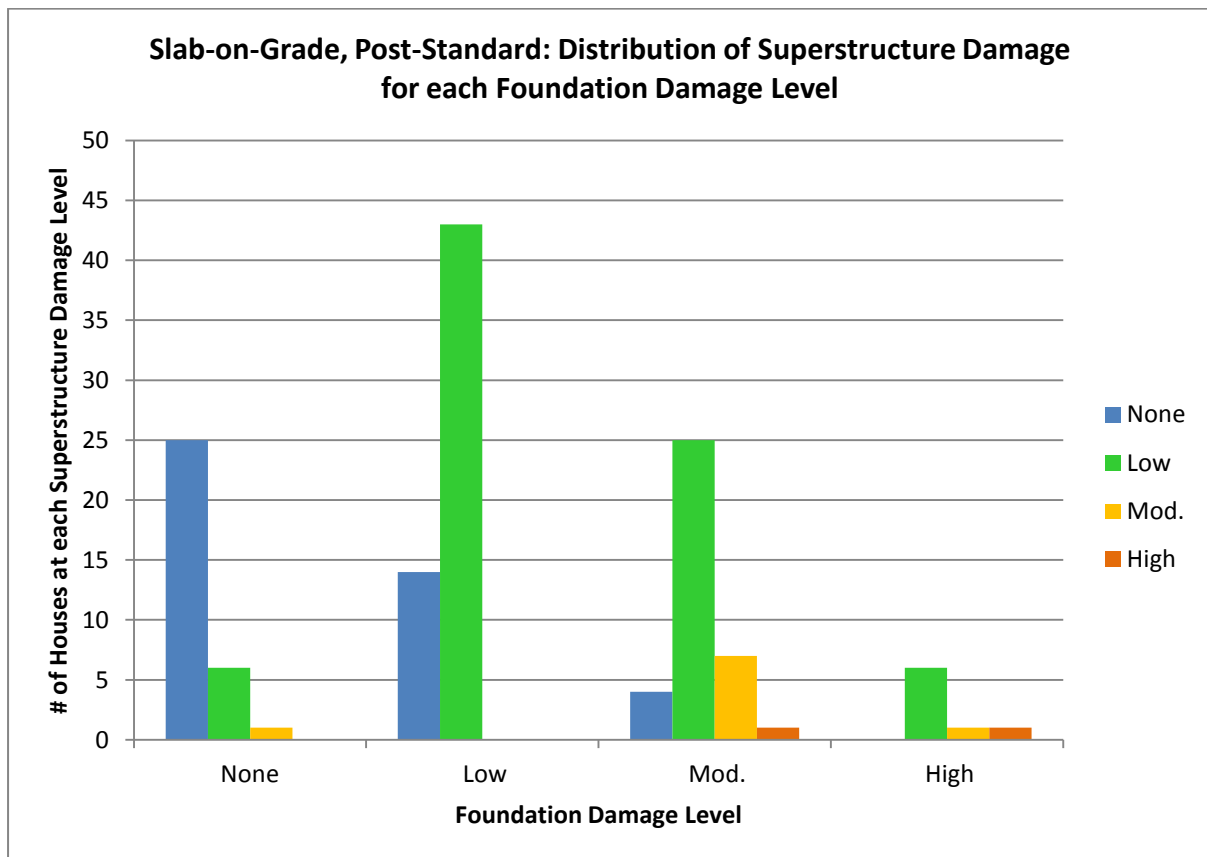


Figure A. 74. Distribution in percentage of damage to house superstructures for differing foundation damage levels for post-standard, slab-on-grade foundations.

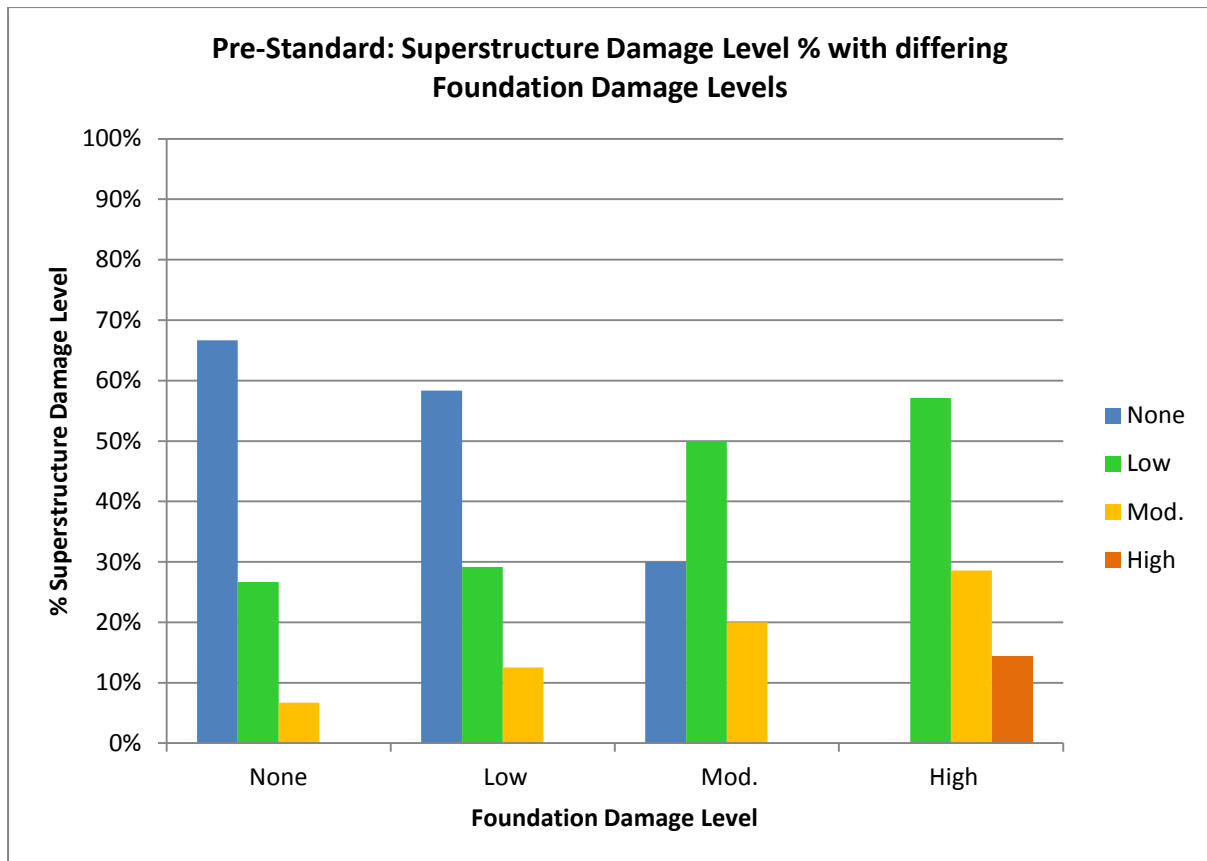


Figure A. 75. Distribution in percentage of damage to house superstructures for differing foundation damage levels for all pre-standard foundations.

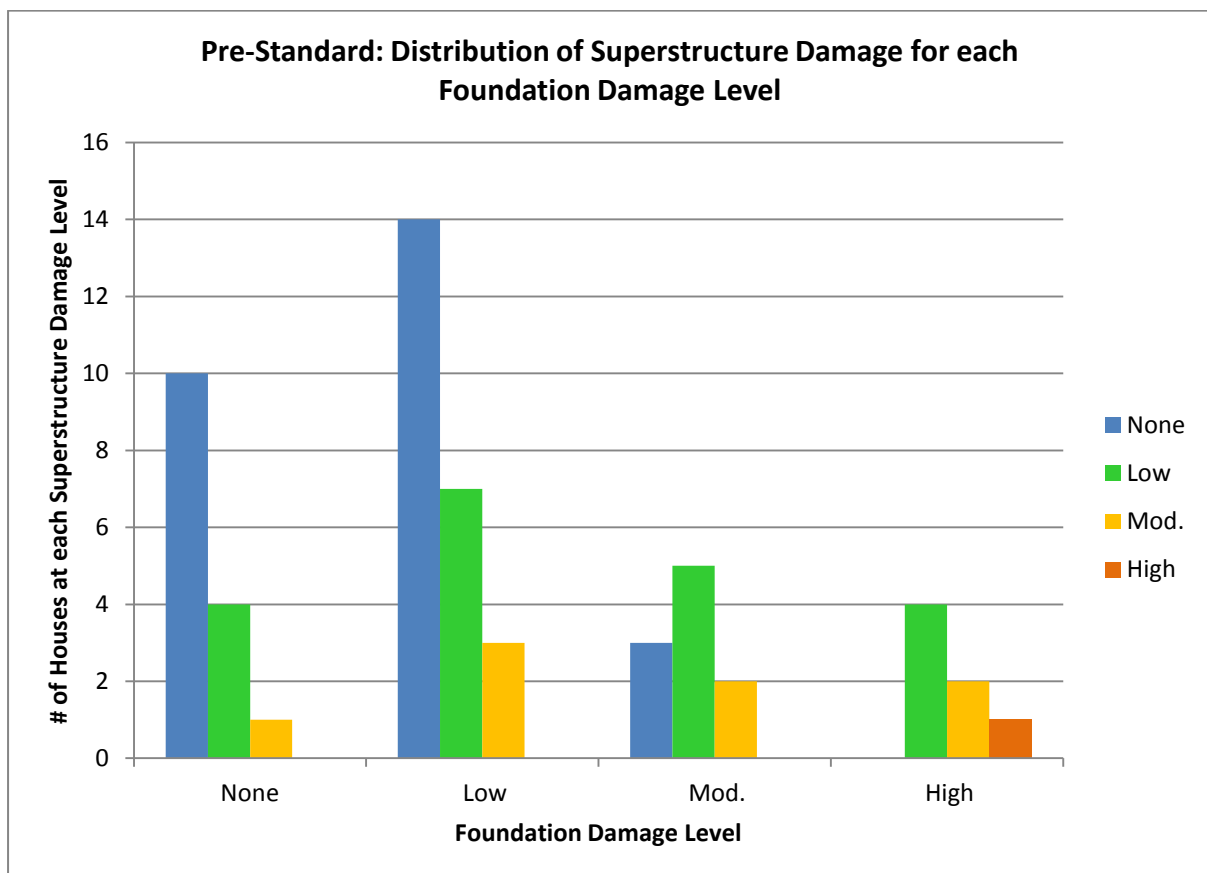


Figure A. 76. Distribution of damage to house superstructures for differing foundation damage levels for all pre-standard foundations.

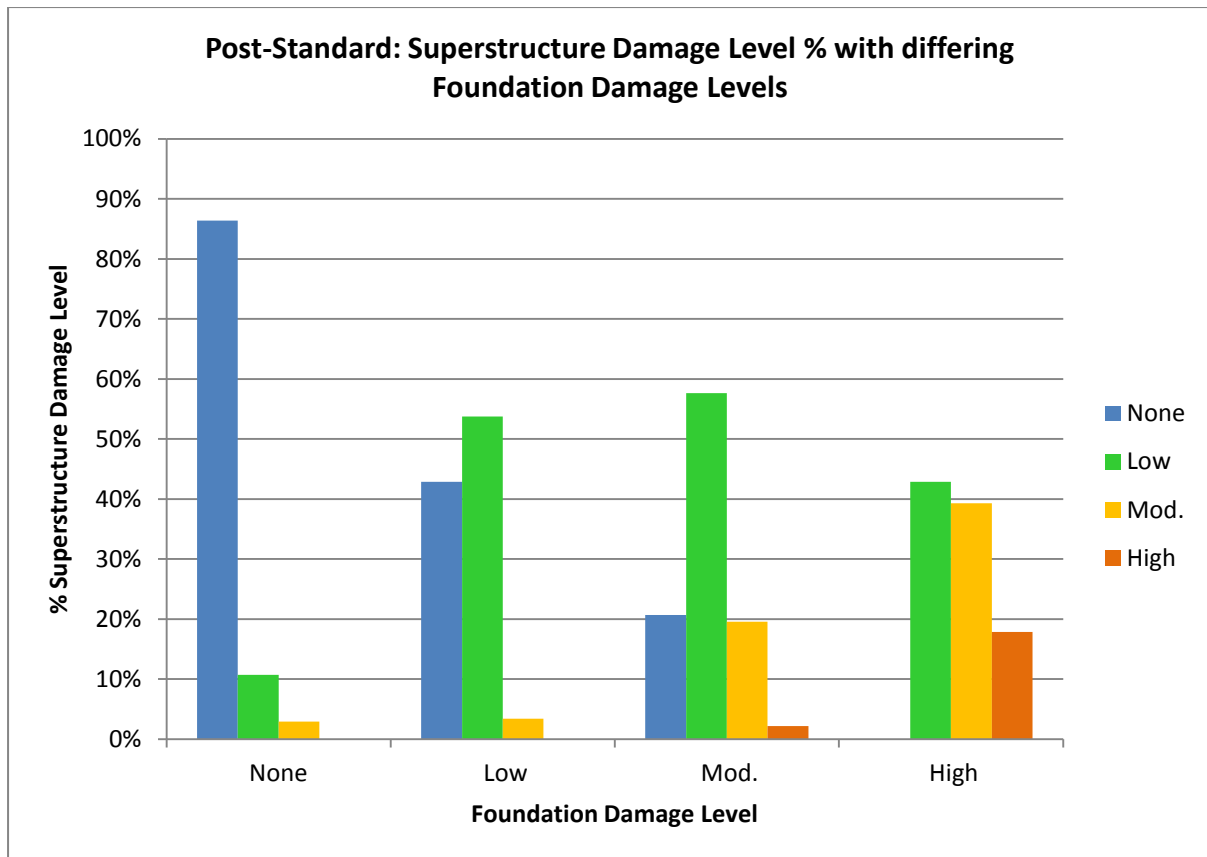


Figure A. 77. Distribution in percentage of damage to house superstructures for differing foundation damage levels for all post-standard foundations.

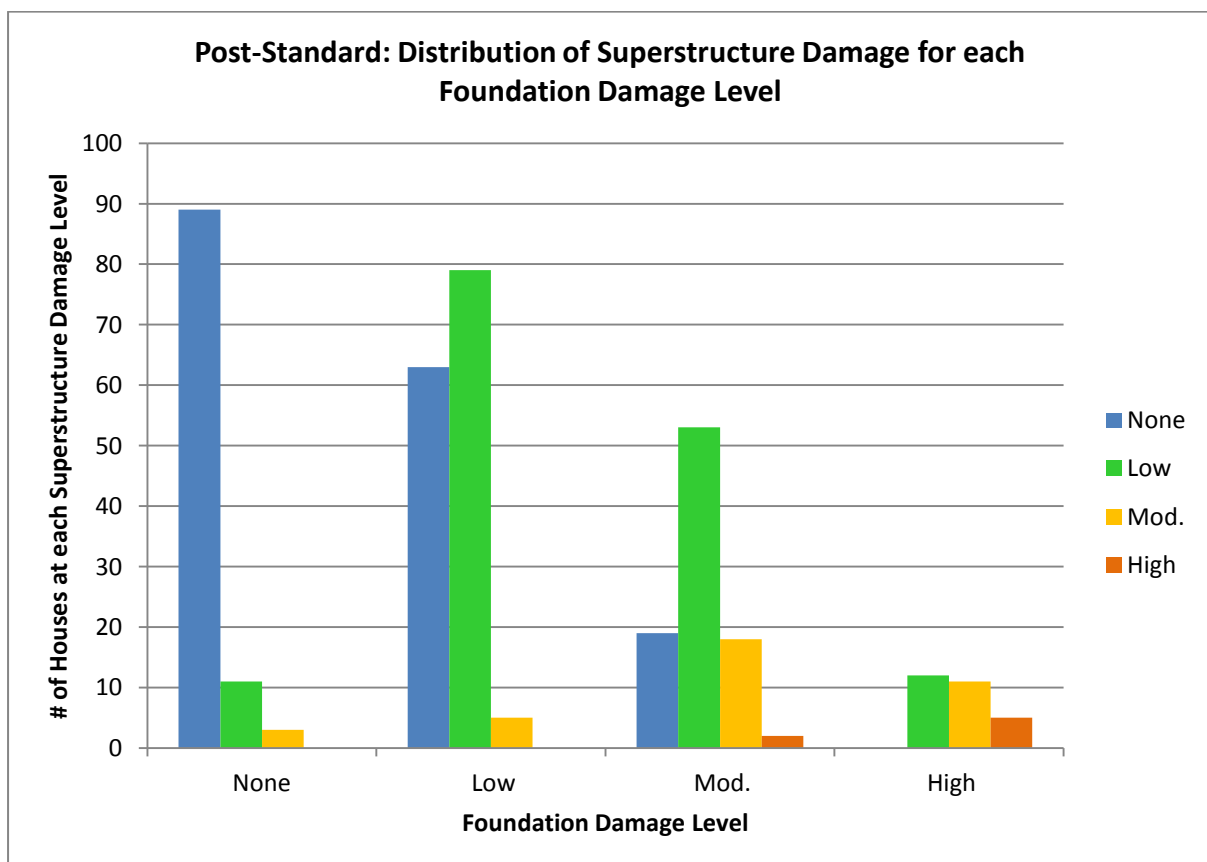


Figure A. 78. Distribution of damage to house superstructures for differing foundation damage levels for all post-standard foundations.

E.4.3: By Building Weight

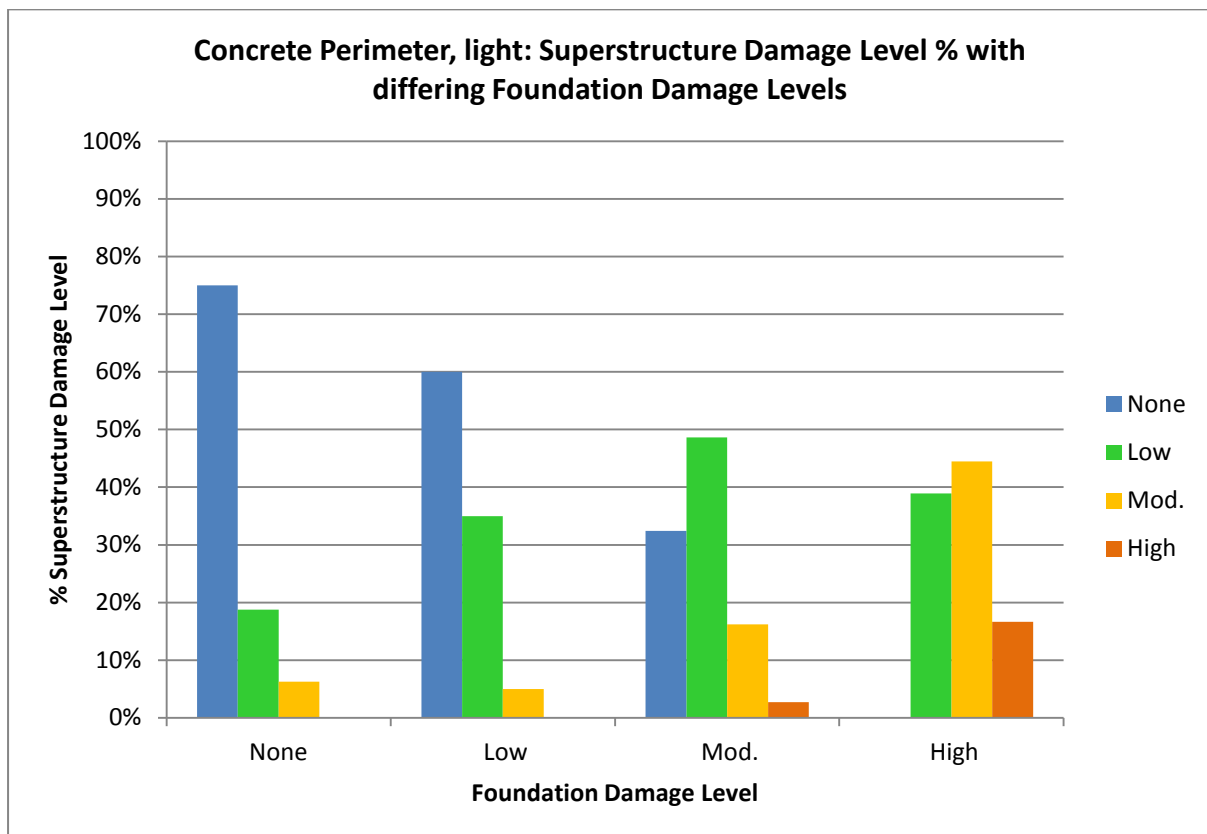


Figure A. 79. Distribution in percentage of damage to house superstructures for differing foundation damage levels for light buildings on concrete perimeter foundations.

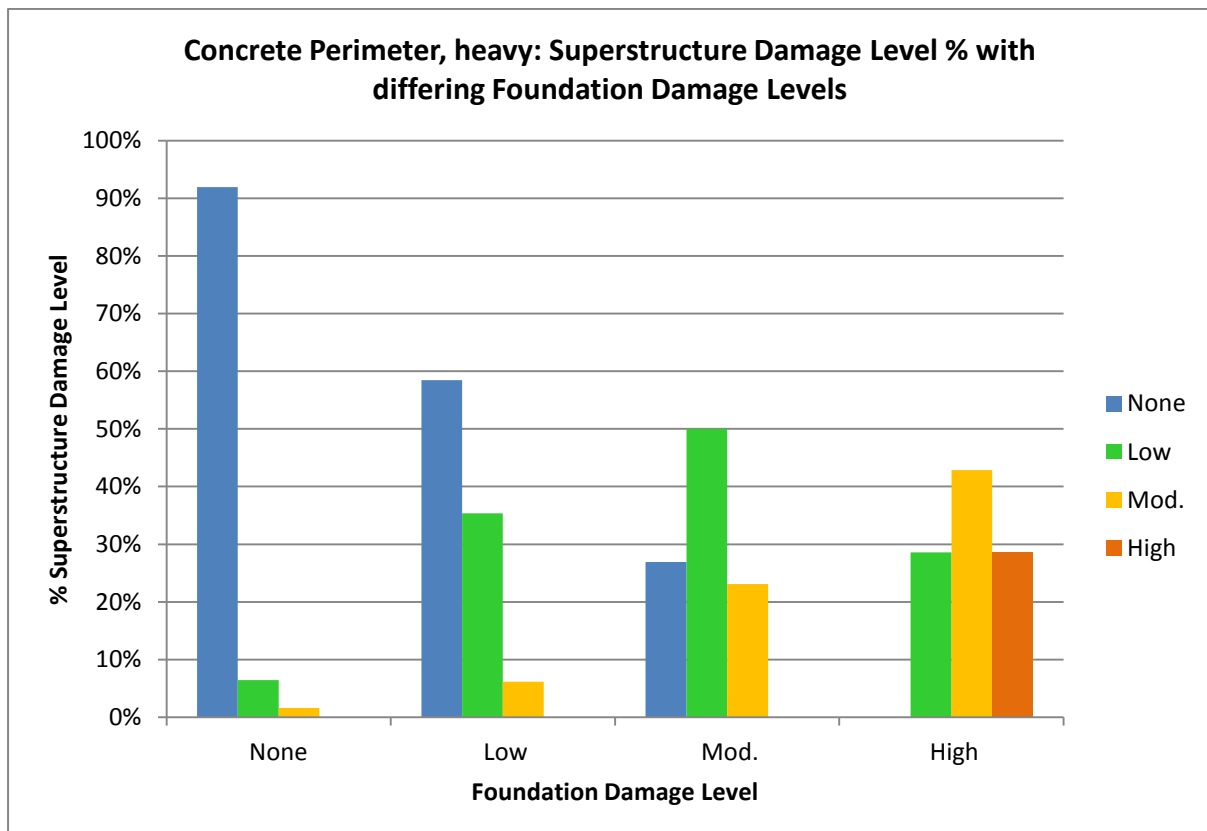


Figure A. 80. Distribution in percentage of damage to house superstructures for differing foundation damage levels for concrete perimeter foundations.

Appendix F: House Inspection Form (March-August 2011)

Address:						
Date:		Examiners:				
Damage Sticker:		<input type="checkbox"/> Green	<input type="checkbox"/> Yellow	<input type="checkbox"/> Red		
Structural Description	Foundation	<input type="checkbox"/> Concrete perimeter footing & short piles <input type="checkbox"/> Concrete slab-on-grade – Reinforced? Y <input type="checkbox"/> , (type: Mesh <input type="checkbox"/> , Rebar <input type="checkbox"/> , N <input type="checkbox"/> , Unknown <input type="checkbox"/> <input type="checkbox"/> Ribraft Slab – Reinforced? Y <input type="checkbox"/> , (type: Mesh <input type="checkbox"/> , Rebar <input type="checkbox"/> , N <input type="checkbox"/> , Unknown <input type="checkbox"/> – Depth of beams: _____. – Beam spacing: _____. <input type="checkbox"/> Piled – Type: Tana 150 SED H5 <input type="checkbox"/> , Hi-stress 90 x 75 <input type="checkbox"/> , 100 ² <input type="checkbox"/> , 150 ² <input type="checkbox"/> – Depth: _____.				
		Cladding	<input type="checkbox"/> Light weight	<input type="checkbox"/> Heavy weight		
		Framing	<input type="checkbox"/> Timber	<input type="checkbox"/> Concrete	<input type="checkbox"/> Steel	<input type="checkbox"/> NV
		Roof	<input type="checkbox"/> Light weight	<input type="checkbox"/> Heavy weight		

Ground Damage	Damage Level				
Liquefaction	None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>	Severe <input type="checkbox"/>
	-	Small SB	Large SB	50 - 200mm	> 200mm
Remarks:					
Lateral Spreading	None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>	Severe <input type="checkbox"/>
	-	< 20mm	20 - 100mm	100 - 300mm	> 300mm
Remarks:					
Foundation Damage	None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>	Severe <input type="checkbox"/>
Type of Damage	Description				

Structural Damage	Damage Level			
External Cladding	None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>
Internal Cladding	None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>
Main Structure	None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>
Structure/Foundation interface	None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>
Surroundings	None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>
Remarks:				

Structural Damage Type	Damage Level			
Stretching	<input type="checkbox"/>	<input type="checkbox"/> Low	<input type="checkbox"/> Moderate	<input type="checkbox"/> High
Hogging	<input type="checkbox"/>	<input type="checkbox"/> Low	<input type="checkbox"/> Moderate	<input type="checkbox"/> High
Dishing	<input type="checkbox"/>	<input type="checkbox"/> Low	<input type="checkbox"/> Moderate	<input type="checkbox"/> High
Racking/Twisting	<input type="checkbox"/>	<input type="checkbox"/> Low	<input type="checkbox"/> Moderate	<input type="checkbox"/> High
Tilting	<input type="checkbox"/>	<input type="checkbox"/> Low	<input type="checkbox"/> Moderate	<input type="checkbox"/> High
Diff. Settlement	<input type="checkbox"/>	<input type="checkbox"/> Low	<input type="checkbox"/> Moderate	<input type="checkbox"/> High
Diff. Displacement	<input type="checkbox"/>	<input type="checkbox"/> Low	<input type="checkbox"/> Moderate	<input type="checkbox"/> High

Detailed Structural Damage Explanation:

Foundation:
(a) settlement of piles; (b) broken piles; (c) house skewed off foundations; (d) cracking in perimeter concrete foundation; (e) cracking in slab.

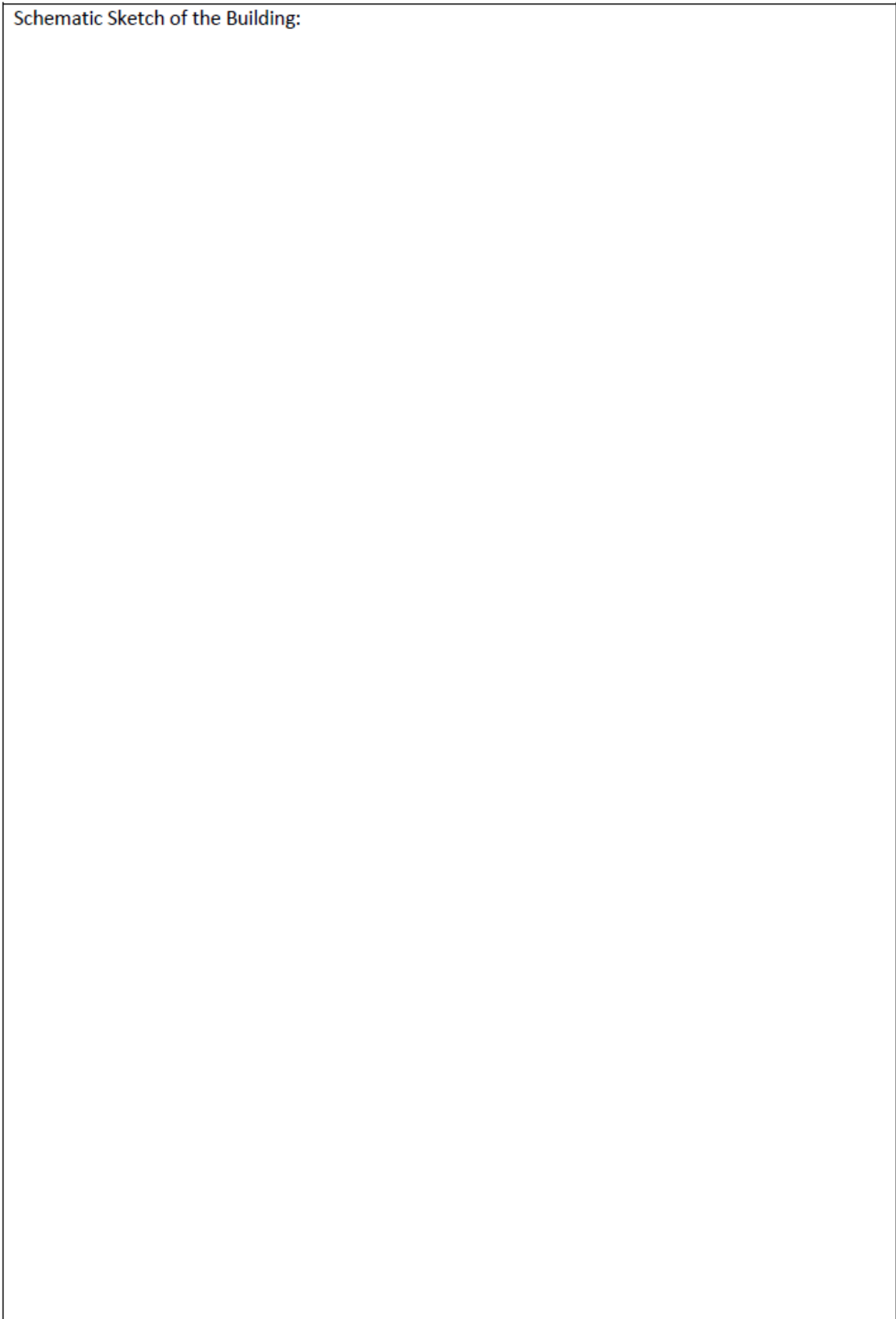
External Partitions:
(a) diagonal cracking; (b) vertical cracking; (c) wall tearing; (d) hairline cracking; (e) gaps opening between window/door frames and wall.

Internal Partitions:
(a) diagonal cracking close to window/door frames; (b) horizontal cracking at ceiling/wall interface; (c) tearing due to lateral spreading

Framing:
(a) nails pulled out between elements of frame; (b) cracking/tearing of frame elements; (c) flexural failure of frame elements

Roof:
(a) buckling of spouting/gutters; (b) buckling of ridges on roof; (c) tearing between rows of tiles/sheets of iron.

Schematic Sketch of the Building:



Appendix G: House Inspection Forms (August 2011 onwards)

Address:							
Owner's Name:							
Contact Details:							
Date:		Examiners:					
Damage Sticker:		<input type="checkbox"/> Green		<input type="checkbox"/> Yellow		<input type="checkbox"/> Red	
Site:		Slope:			Adjacent to watercourse?		
		Flat <input type="checkbox"/>	Sloping <input type="checkbox"/>		Yes <input type="checkbox"/>	No <input type="checkbox"/>	
General Building							
Age band:		Pre 1930 <input type="checkbox"/>	1930-59 <input type="checkbox"/>	1960-79 <input type="checkbox"/>	1980-99 <input type="checkbox"/>	Post 2000 <input type="checkbox"/>	
Stories:		1 <input type="checkbox"/>	2 <input type="checkbox"/>	Split-level <input type="checkbox"/>			
Footprint area:		<100m ² <input type="checkbox"/>	100-200m ² <input type="checkbox"/>	200-300m ² <input type="checkbox"/>	>300m ² <input type="checkbox"/>		
Plan shape:		Rectangle <input type="checkbox"/>	"L" shape <input type="checkbox"/>	"T" shape <input type="checkbox"/>	Complex <input type="checkbox"/>	Other <input type="checkbox"/>	
Structural Description	Foundation	<input type="checkbox"/> Concrete perimeter footing & short piles <input type="checkbox"/> Concrete slab-on-grade - Reinforced? Y <input type="checkbox"/> , (type: Mesh <input type="checkbox"/> , Rebar <input type="checkbox"/>) N <input type="checkbox"/> , Unknown <input type="checkbox"/> <input type="checkbox"/> Ribraft Slab - Reinforced? Y <input type="checkbox"/> , (type: Mesh <input type="checkbox"/> , Rebar <input type="checkbox"/>) N <input type="checkbox"/> , Unknown <input type="checkbox"/> - Depth of beams: _____ - Beam spacing: _____ <input type="checkbox"/> Piled - Type: Tana 150 SED H5 <input type="checkbox"/> , Hi-stress 90 x 75 <input type="checkbox"/> , 100 ² <input type="checkbox"/> , 150 ² <input type="checkbox"/> - Depth: _____					
		Floor	Type:	Concrete Slab <input type="checkbox"/>	Suspended Timber <input type="checkbox"/>	Sheet flooring <input type="checkbox"/>	Timber strip floor <input type="checkbox"/>
			Height:	<200mm <input type="checkbox"/>	200-400 <input type="checkbox"/>	400-600 <input type="checkbox"/>	>600 <input type="checkbox"/>
		Framing	Framed <input type="checkbox"/> (T <input type="checkbox"/> S <input type="checkbox"/>)	U.R.M. <input type="checkbox"/>	Concrete <input type="checkbox"/>	Panelised <input type="checkbox"/>	Other <input type="checkbox"/>
	Cladding	Weatherboard <input type="checkbox"/>		Brick vnr <input type="checkbox"/>	Stone vnr <input type="checkbox"/>	Conc.Blk vnr <input type="checkbox"/>	
		Stucco <input type="checkbox"/>	Solid brick <input type="checkbox"/>	Ply sheet <input type="checkbox"/>	FC sheet <input type="checkbox"/>	Other <input type="checkbox"/>	
	Roof	Heavy tiles <input type="checkbox"/>	Metal tiles <input type="checkbox"/>	Iron <input type="checkbox"/>	Shingles <input type="checkbox"/>	Other <input type="checkbox"/>	
Ground Damage		Damage Level					
Liquefaction		-	Small SB	Large SB	50 - 200mm	> 200mm	
September		None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>	Severe <input type="checkbox"/>	
February		None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>	Severe <input type="checkbox"/>	
Eq. 3:		None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>	Severe <input type="checkbox"/>	
Eq. 4:		None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>	Severe <input type="checkbox"/>	
Remarks:							
Lateral Spreading		-	<20mm	20-100mm	100-300mm	> 300mm	
September		None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>	Severe <input type="checkbox"/>	
February		None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>	Severe <input type="checkbox"/>	
Eq. 3:		None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>	Severe <input type="checkbox"/>	
Eq. 4:		None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>	Severe <input type="checkbox"/>	
Remarks:							

General Building		Damage Level		
Stretching	<input type="checkbox"/>	<input type="checkbox"/> Low	<input type="checkbox"/> Moderate	<input type="checkbox"/> High
Hogging	<input type="checkbox"/>	<input type="checkbox"/> Low	<input type="checkbox"/> Moderate	<input type="checkbox"/> High
Dishing	<input type="checkbox"/>	<input type="checkbox"/> Low	<input type="checkbox"/> Moderate	<input type="checkbox"/> High
Racking/Twisting	<input type="checkbox"/>	<input type="checkbox"/> Low	<input type="checkbox"/> Moderate	<input type="checkbox"/> High
Tilting	<input type="checkbox"/>	<input type="checkbox"/> Low	<input type="checkbox"/> Moderate	<input type="checkbox"/> High
Diff. Settlement	<input type="checkbox"/>	<input type="checkbox"/> Low	<input type="checkbox"/> Moderate	<input type="checkbox"/> High
Diff. Displacement	<input type="checkbox"/>	<input type="checkbox"/> Low	<input type="checkbox"/> Moderate	<input type="checkbox"/> High

Surroundings						
Decks:						
Settled	None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>	Severe <input type="checkbox"/>	
Tilting	None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>	Severe <input type="checkbox"/>	
Ruptured	None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>	Severe <input type="checkbox"/>	
Separated	None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>	Severe <input type="checkbox"/>	
Concrete Paving:	None	<10%	10-20%	20-50%	50-75%	75-100%
Cracking	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Separation	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Tilting	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Settlement	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Steps:	Cracking <input type="checkbox"/>	Separation <input type="checkbox"/>	Tilting <input type="checkbox"/>	Settlement <input type="checkbox"/>		
Extent						
Fences/Walls:	None	<10%	10-20%	20-50%	50-75%	75-100%
Leaning	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Ruptured	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Fallen	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Retaining Walls:	None	<10%	10-20%	20-50%	50-75%	75-100%
Leaning	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Ruptured	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Fallen	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>

Comments:

Structural Damage	Damage Level					
External Cladding	Masonry <input type="checkbox"/>	Stucco <input type="checkbox"/>	Weatherboard <input type="checkbox"/>	Other <input type="checkbox"/>		
Damage type:	None	<10%	10-20%	20-50%	50-75%	75-100%
No Damage	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Diagonal cracking	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Vertical cracking	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Horizontal cracking	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Detached/unstable	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Fallen	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Corners separating	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Windows	None	<10%	10-20%	20-50%	50-75%	75-100%
No Damage	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Jammed	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Broken	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Doors	None	<10%	10-20%	20-50%	50-75%	75-100%
No Damage	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Jammed	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Broken	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Internal Cladding	None	<10%	10-20%	20-50%	50-75%	75-100%
Cracking at corners	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Joint cracks	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Separation	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Nail popping	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Buckling	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Roof	None	<10%	10-20%	20-50%	50-75%	75-100%
Mis-alignment	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Buckling	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Hips loose	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Popped fixings	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Joint cracks	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Tiles dislodged	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Comments:						

Foundation Damage	None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>	Severe <input type="checkbox"/>
Slab	<input type="checkbox"/> Undamaged		<input type="checkbox"/> Unable to check		
Separation at joints:	Y <input type="checkbox"/>	N <input type="checkbox"/>	Width:	Min.	Max.
Cracking:	1	2	3	4	5
Width (mm)					
Length (m)					
Offset (mm)					

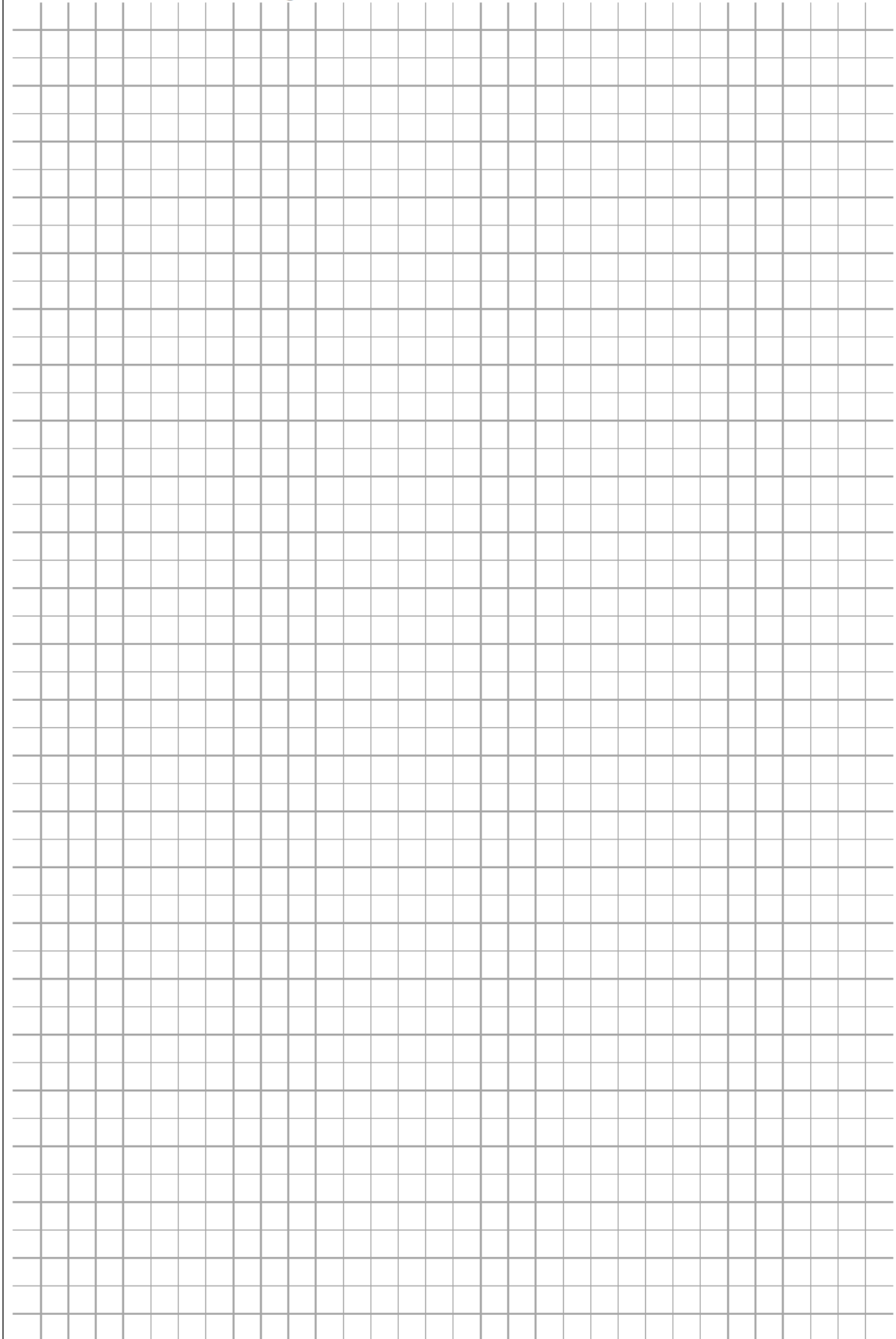
(OR)

Foundation Damage	None <input type="checkbox"/>	Low <input type="checkbox"/>	Moderate <input type="checkbox"/>	High <input type="checkbox"/>	Severe <input type="checkbox"/>
Concrete Perimeter	<input type="checkbox"/> Undamaged		<input type="checkbox"/> Floor detached from perimeter		
Cracking:					
Side	1	2	3	4	5
Name					
# cracks					
Size range					
Mis-alignment?					
'Piles':	Vis	NV	None	<10%	10-20%
Settled			<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Tilting			<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>

Global Settlement Measurements			
Position	Dist. betw. points	Settlement	Description

Type of Damage	Description

Schematic Sketch of the Building:



Appendix H: Permission Form

Date: ____/____/____.

Address: _____.

Dear Owner/Resident,

As part of the research outlined in the attached letter, I wish to inspect and assess your house for damage due to the September 2010 and/or February 2011 earthquakes.

As part of this process, I will take photos and measurements of the damage to your house/property. For example: measuring crack widths with a ruler/tape measure; estimating the tilt of the structure with a smart level; photographing damage. The inspection will not damage your property, and the information gathered will be used solely for research purposes and will not influence any EQC or insurance claims you have.

Please indicate whether:

(tick)

- ☐ - You are welcome to do a full inspection when I am at home, which will be:
(please indicate days/times that suit)

_____.
- ☐ - I give you permission to complete an external inspection in my absence (if you cannot be here for the inspection, it would be much appreciated if you would fill out the attached questionnaire).
- ☐ - I do not wish to have my house inspected.

Please leave this page, completed, in your letterbox for me to collect. You may keep the attached letter.

Thank you for your time,
Yours sincerely,

Duncan Henderson
Masters Candidate
Department of Civil and Natural Resources Engineering
University of Canterbury, Private Bag 4800, Christchurch 8140
Ph: 027 636 4518
Email: duncan.henderson@pg.canterbury.ac.nz

Appendix I: Homeowner Questionnaire

Address:			
Name:		Date:	

- 1 Was your property subject to Liquefaction/other land damage during the following events? (please tick all that apply)

	Liquefaction	Ground Cracking
10th September, 2010		
19 th October, 2010 (Aftershock)		
26 th December, 2010 (Aftershock)		
22nd February, 2011		
16 th April, 2011 (Aftershock)		

- 2 Please comment on the damage to your house and property as a result of the two earthquakes and three major aftershocks.

- a) Property (land)

eg: - Liquefaction - extent, size & depth of sand volcanoes
 - Cracks in lawn/garden - width, length, depth

Date of Quake	Type of damage	Extent/description of damage

- b) Foundation of house

eg: - crack in foundation slab - width, length of this
 - separation of house from foundation - extent of movement
 - tilting of foundation/house

} have these increased in subsequent events?

Date of Quake	Extent/description of damage

c) Structure of house

eg: - cracking in walls - location, direction, size of cracks

- tilting of house - direction (i.e. low point, high point)

have these increased in

subsequent events?

Date of Quake	Extent/description of damage

Thank you for your time completing this survey. If you wish to discuss your answers or the nature of the

research involved, please feel free to contact me: duncan.henderson@pg.canterbury.ac.nz

Appendix J: Official Letter to Residents

Department of Civil and Natural Resources Engineering
College of Engineering
Tel: +64 3 364 2250, Fax: +64 3 364 2758, www.civil.canterbury.ac.nz



12 May 2011

Dear Owner/Resident,

This letter is to introduce the activities that Duncan Henderson of the University of Canterbury is undertaking as part of research relating to the September 2010 Darfield Earthquake and February 2011 Christchurch Earthquake. As part of a Masters' thesis at the University of Canterbury, he is studying the land damage as a result of the two earthquakes (liquefaction and lateral spreading) and the effect this damage has had on the foundations of structures, and in turn the structures themselves, in particular residential houses. He is looking in detail at the performance of four different foundation types (concrete perimeter with short piles, slab-on-grade, rib-raft/waffle slabs and piles) and how these performed during the earthquake with a view to facilitating better design of houses for earthquake performance. The output from these studies will provide invaluable information both for the recovery process and will aide in the development of improved design codes and hence more resilient infrastructure for Canterbury and New Zealand.

We have no connection with the Earthquake Commission, District or Regional Councils, or individual companies, however, we collaborate with various parties and keep others aware of the work we are doing. All the data collected will be available to the public after detailed analysis and interpretation. We assure you that all data will be used solely for research purposes and benefit to the wider public.

We thank you for your cooperation and wish you all the best.

Yours sincerely,

Associate Professor Misko Cubrinovski
Deputy Head of Department
Department of Civil and Natural Resources Engineering
University of Canterbury, Private Bag 4800, Christchurch 8140
Ph: +64 3 364 2251; Fax: +64 3 364 2758
Email: misko.cubrinovski@canterbury.ac.nz

Appendix K: Data Recorded

#	Damage Measure	Units & Range	Measured of Judged?	Positives and Negatives	Used?
1	Average Global Settlement	mm +/- any	Measured with judgement	Global Settlement was not recorded on most properties, as it was often hard to identify whether or not it had occurred. Only a few properties had obvious settlement relative to the surrounding land. In most cases, the surrounding land settled with the properties.	No
2	Overall Differential Settlement	mm/ slope any	Measured	This is a good measure of the overall deflection of the house, caused by ground failure (either through liquefaction uplift or sinking, lateral spreading or bearing failure)	Yes
3	Maximum Differential Settlement	mm/ slope any	Measured	This is a good measure of the distortion of the foundation, whether through hogging, dishing, or uniform tilting, and indicates how badly damaged the foundation is.	Yes
4	Average Floor Tilt	° any	Measured & combined	This is an unrealistic representation of the floor tilts taken with a spirit level, as any large tilts recorded are diluted by the number of smaller tilts often recorded on any foundation that is not tilting uniformly	No
5	Maximum Floor Tilt	° any	Measured	This value is a good measure of the totally local distortion of the foundation, and so is a good indicator of how easily and by how much the foundation can be distorted by any overall damage	Yes
6	Overall Foundation Damage	N/A 0 – 4	Judged	This measure is judged by the inspector, so has room for error, but with only 5 options, is generally a good indicator of the overall performance of the foundation	Yes
7	Structural Damage Index	N/A 0 – 21	Judged & Combined	This is a combination of seven structural damage factors, all of which have to be judged by the inspector. As they are combined, this artificially spreads out the data where maybe it shouldn't be, and can increase the effect of any errors in judgement. Some of the factors are also somewhat repeats of each other (i.e. hogging and dishing)	No

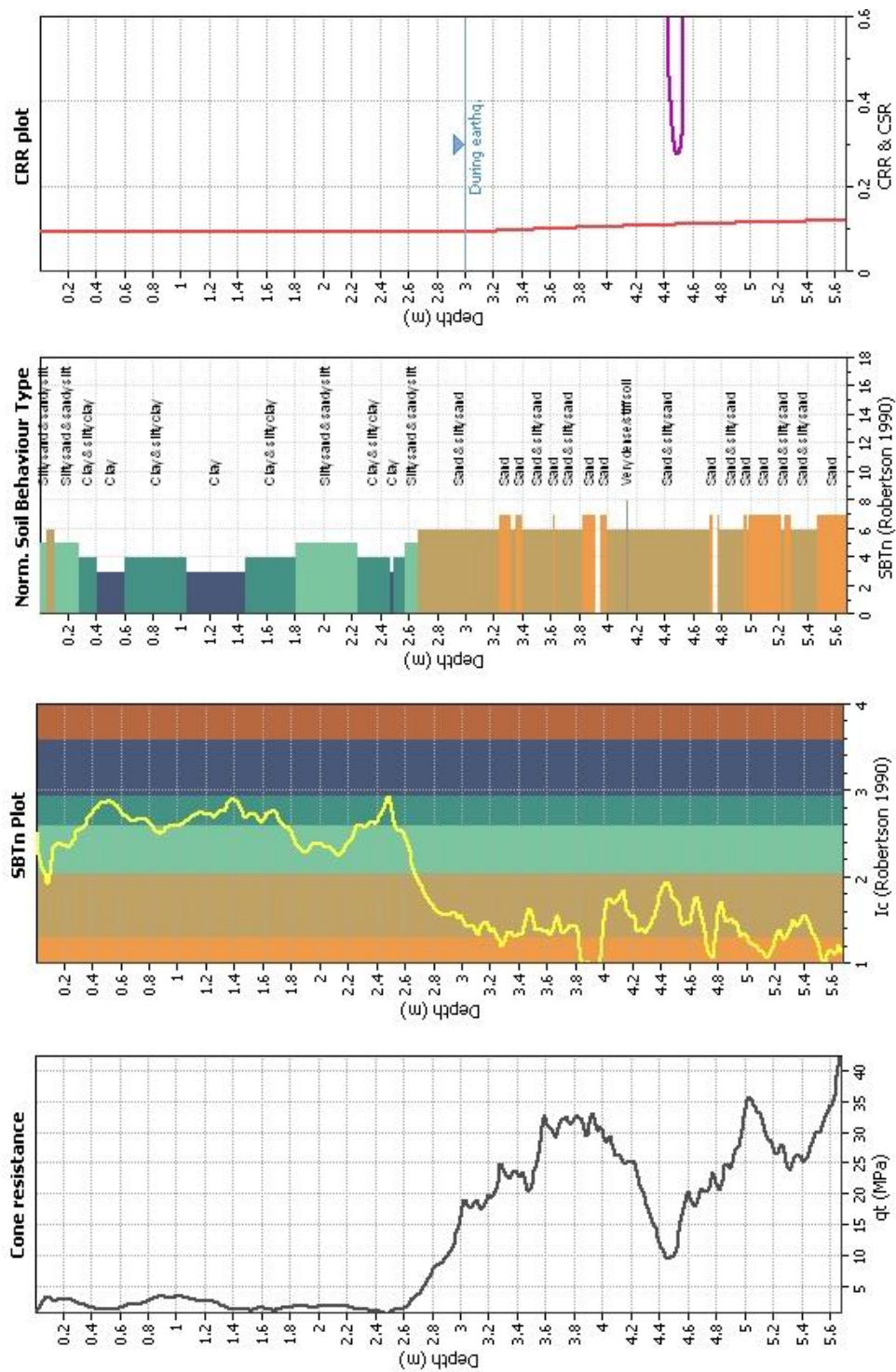
8	Overall Structural Damage	N/A 0 – 4	Judged	Like the Overall Foundation Damage, although judged by the inspector, it is a good indicator of the overall damage level of the superstructure. More so in this case as it wasn't possible to find anything that could be consistently measured on each house to represent the structural damage level	Yes
9	Total Structural Damage	N/A 0 – 120	Judged & Combined	Like the Structural Damage Index, this is a combination of judged factors on various specifics of superstructure damage. However in this case there are 20 different factors which are combined, producing a large spread in the data which isn't really warranted. This also allows for a large amount of error.	No
10	Maximum crack width	mm any	Measured	This measure doesn't really give much information about the overall performance of the foundation. The largest crack may only be 5mm, but there could be 20 of them adding up to 70mm displacement, compared to a foundation where there is only one 40mm crack.	No
11	Total crack width	mm any	Measured	This is a good measure of the effect of lateral spreading on perimeter houses, as the total stretch of the foundation is taken into account. Foundations can crack just from liquefaction, but often will not separate more than ~5mm per crack	Yes?

Appendix L: Summary of Structural Features

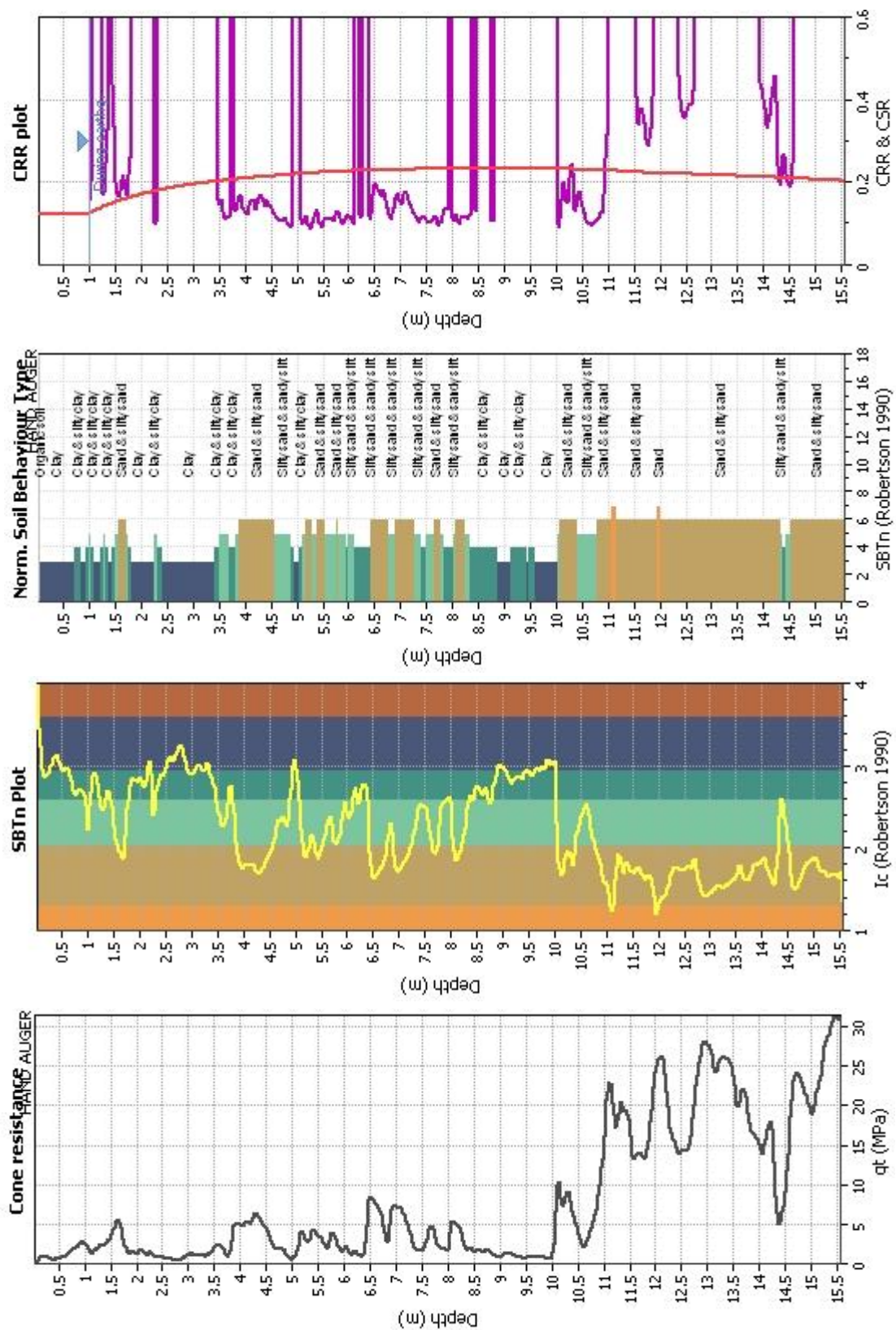
Structural Feature		Concrete Perimeter	Slab-on-Grade	RibRaft	Piled
Age	Pre-1930	25			
	1930-59	7	1		
	1960-79	9	3	1	
	1980-99		10		6
	Post-2000	1	8	33	24
	Unknown	6	10	5	3
Number of Storeys	1	44	22	8	14
	2	3	18	28	16
	Other	2	2	2	3
Floor Plan Size	<100m ²	1	8	10	11
	100-200m ²	42	25	21	13
	200-300m ²	6	7	6	4
	>300m ²			1	2
Floor Plan Shape	Rectangular	25	19	14	15
	“L” shaped	13	7	10	4
	“T” shaped	2	6	12	1
	Complex	8	9	3	12
	Other	1	1		
Floor Height	<200mm		37	34	29
	200-400mm	11	5	5	3
	400-600mm	34	1		1
	>600mm	4			
Exterior Wall Cladding	Weatherboard	31	1	6	3
	Stucco	4	11	15	8
	Brick	3	16	3	12
	Stone	4	5	5	3
	Concrete Block	3	3		
	Combination	4	6	7	6
	Other			2	
Roof Cladding	Iron	35	33	36	28
	Metal Tiles	1	3	2	3
	Heavy Tiles	12	6	1	1
	Other	1			1

Appendix M: CPT and Liquefaction Analysis Results

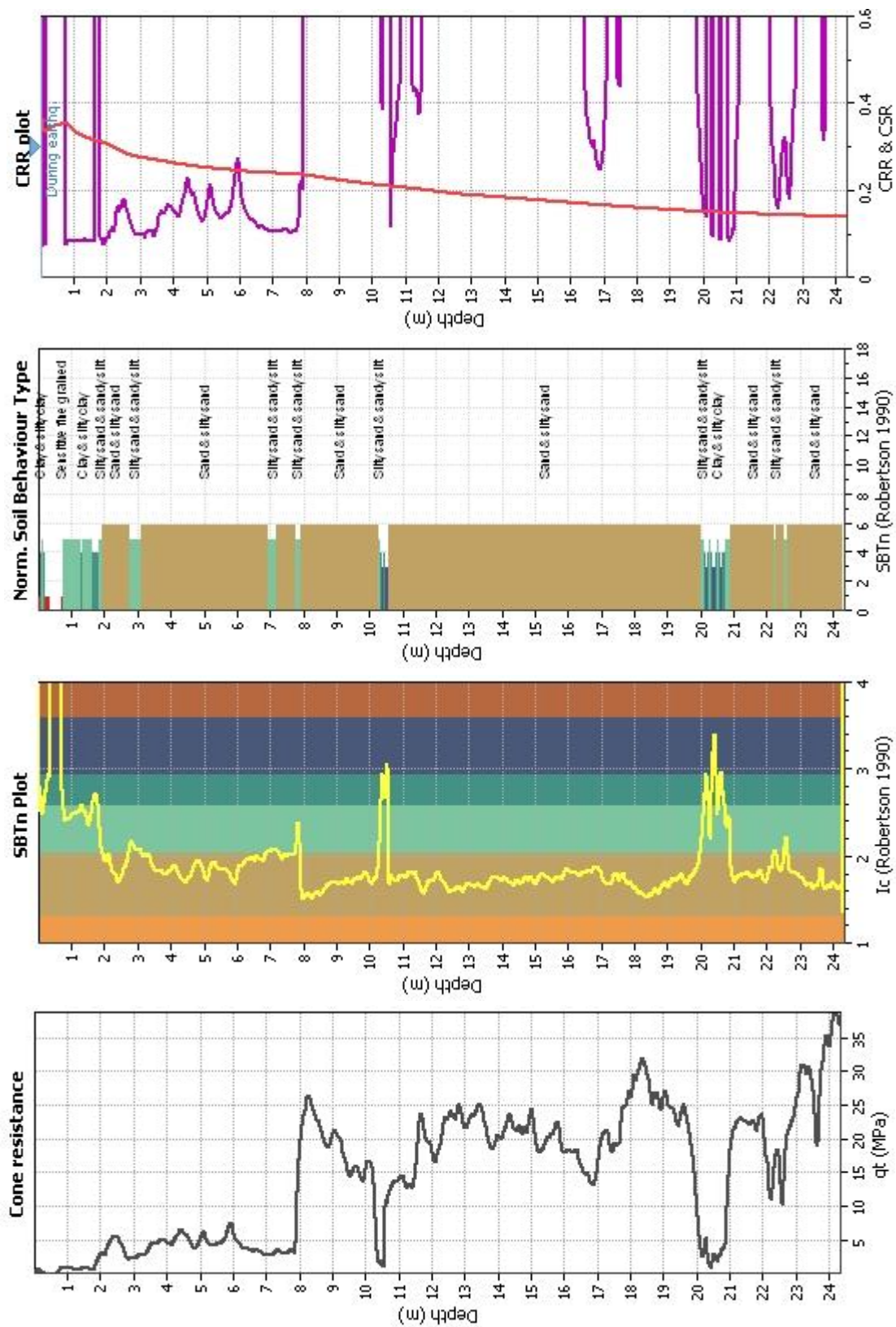
M.1: Ilam (CPT 24894)



M.2: Merivale (CPT 6583)

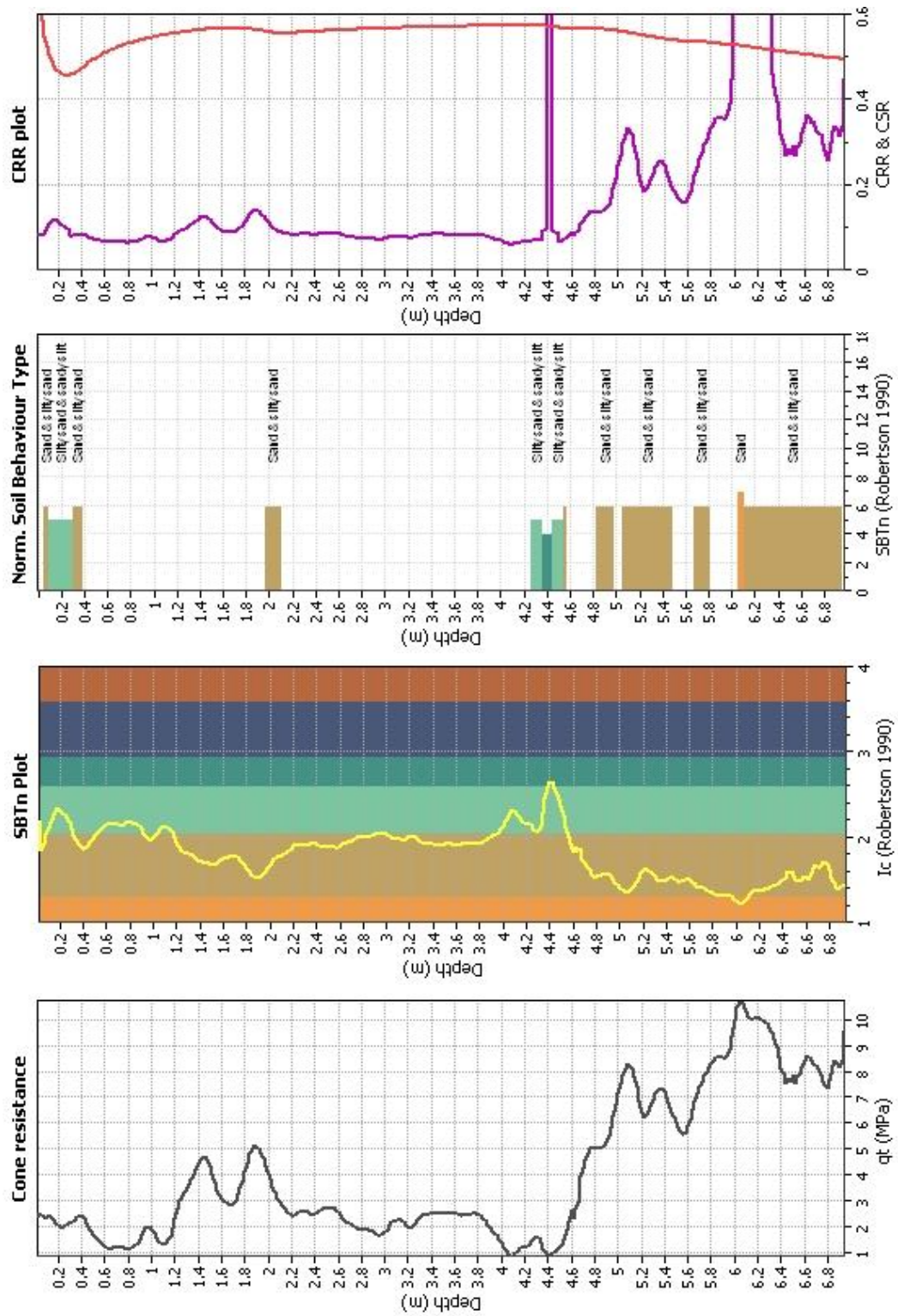


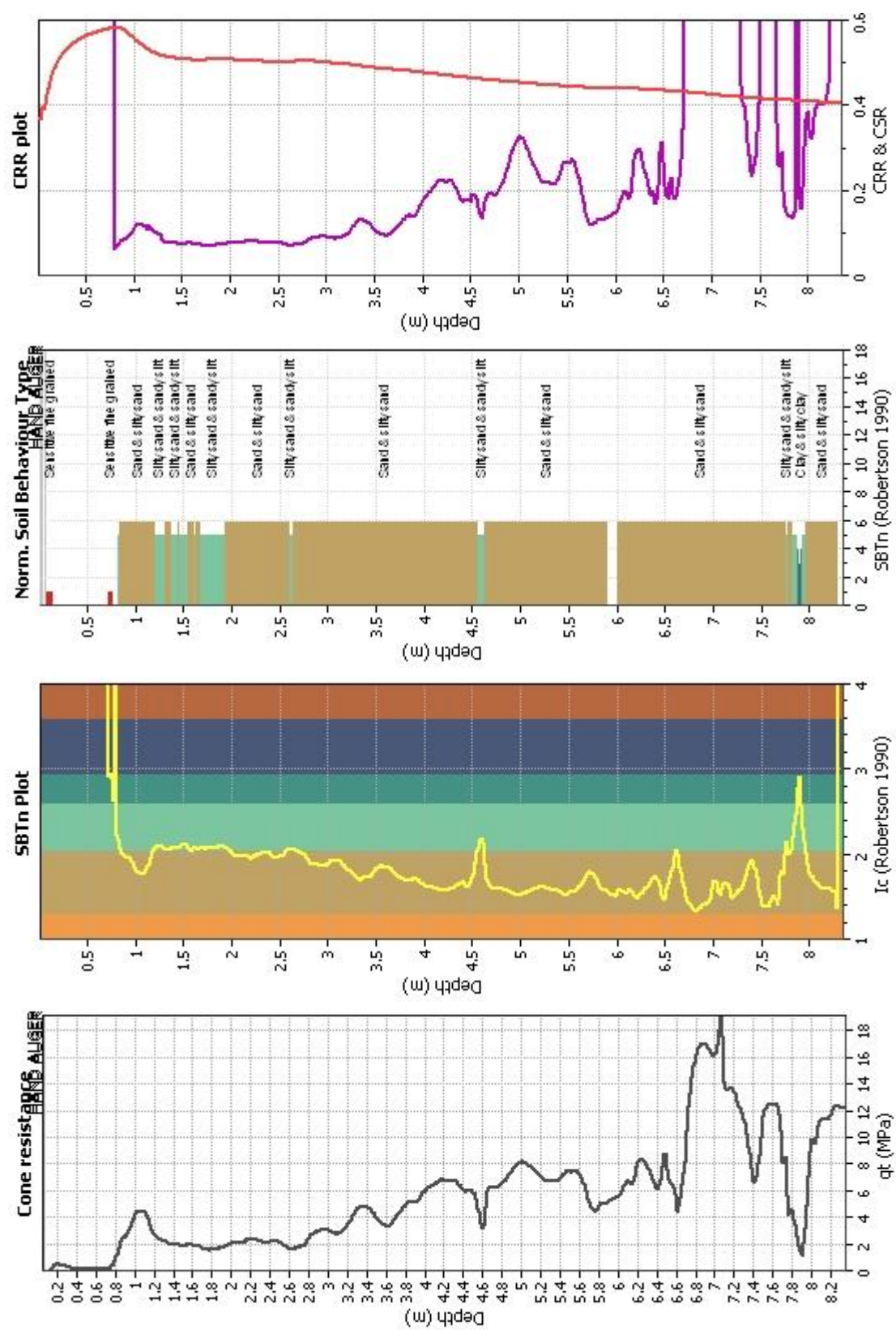
M.3: Horseshoe Lake, Burwood (CPT 272)



M.4: Dallington (CPT 1079 & 1087)

1079





Appendix N: 22nd February 2011 Analysis Results

N.1: Max. Liquefaction Severity vs. Foundation Damage Parameters

N.1.1: Concrete Perimeter

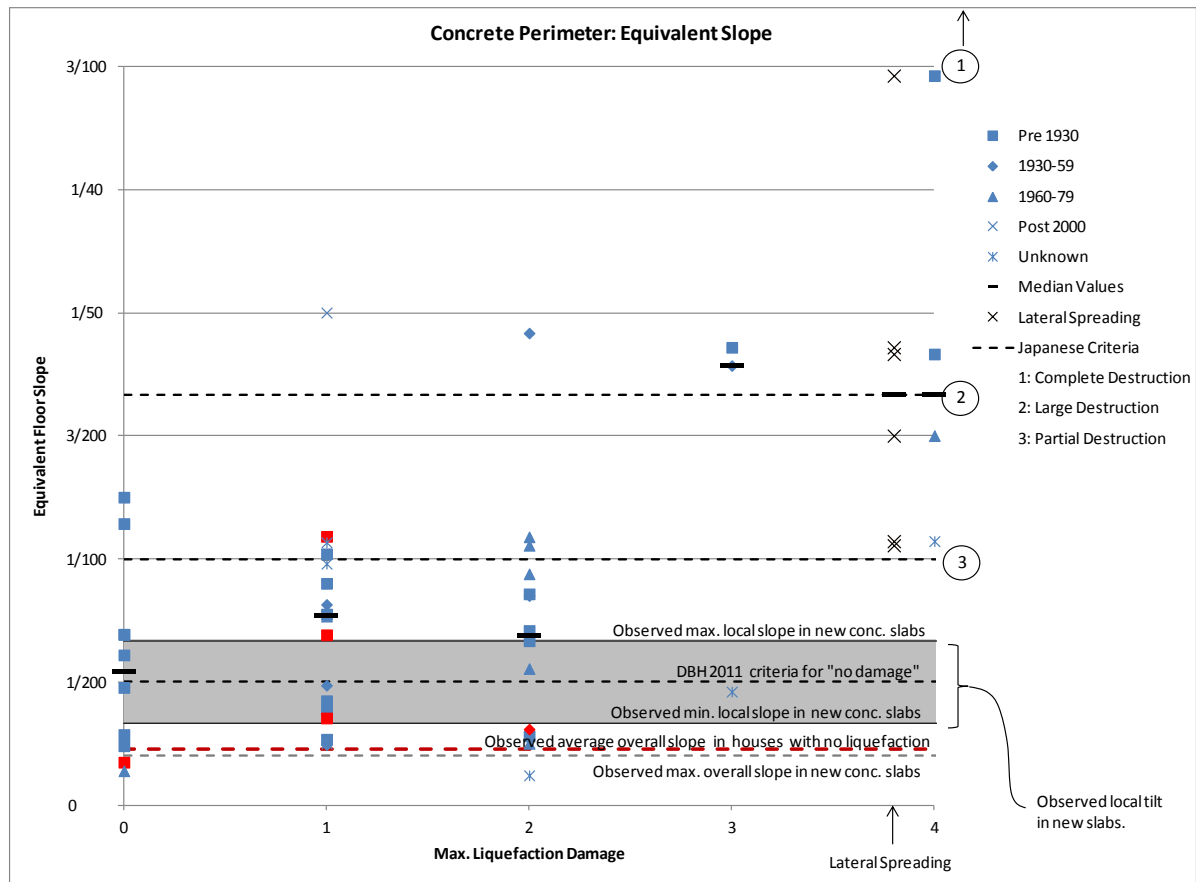


Figure A. 81. Equivalent floor slope vs. maximum liquefaction damage for concrete perimeter foundations, showing lateral spreading data and no damage references.

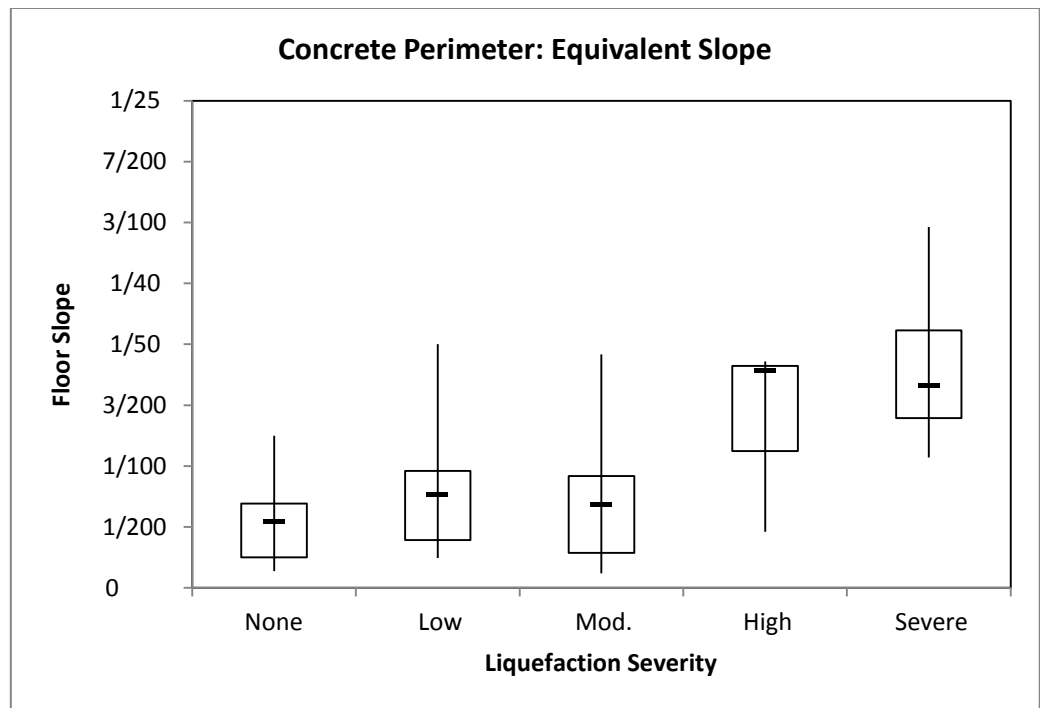


Figure A. 82. Box & Whisker plot for the equivalent slope of concrete perimeter foundations for each level of liquefaction severity.

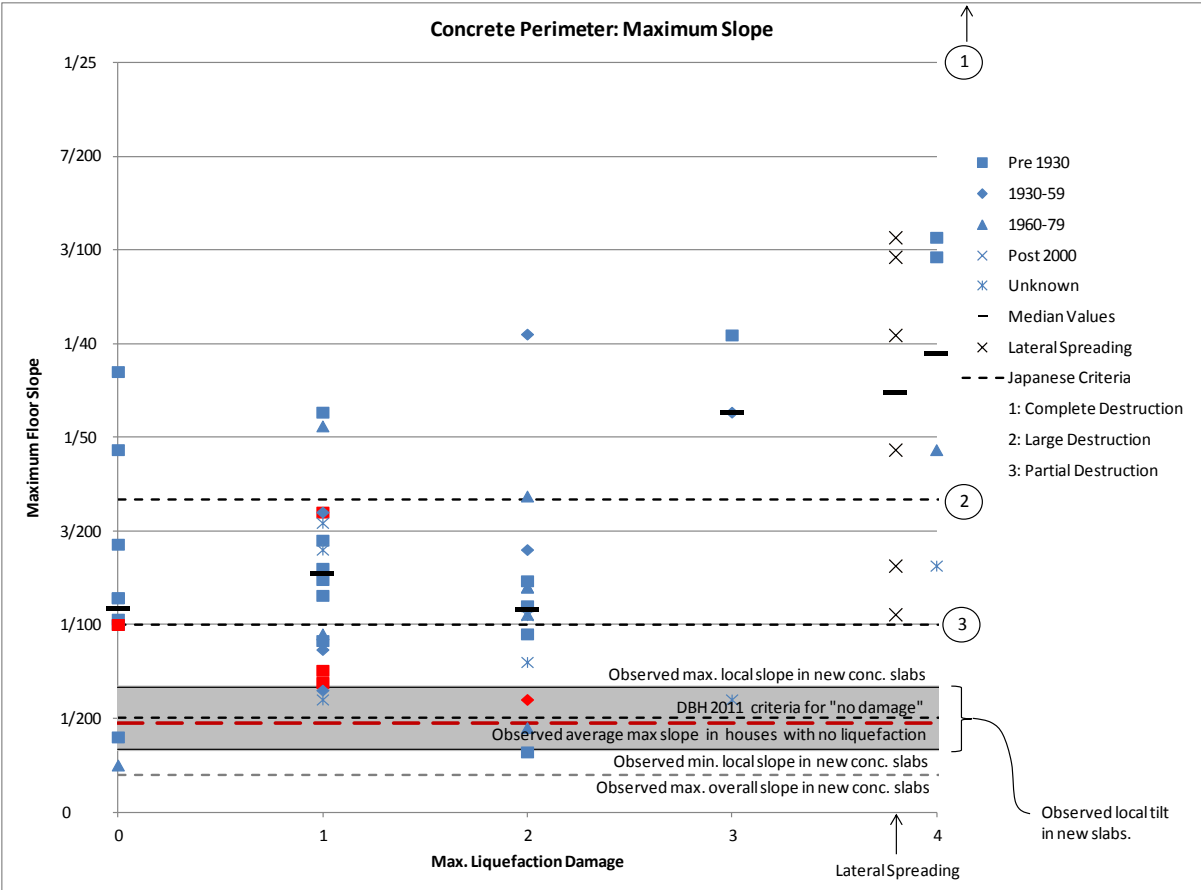


Figure A. 83. Maximum floor slope vs. maximum liquefaction damage for concrete perimeter foundations, showing lateral spreading data and no damage references.

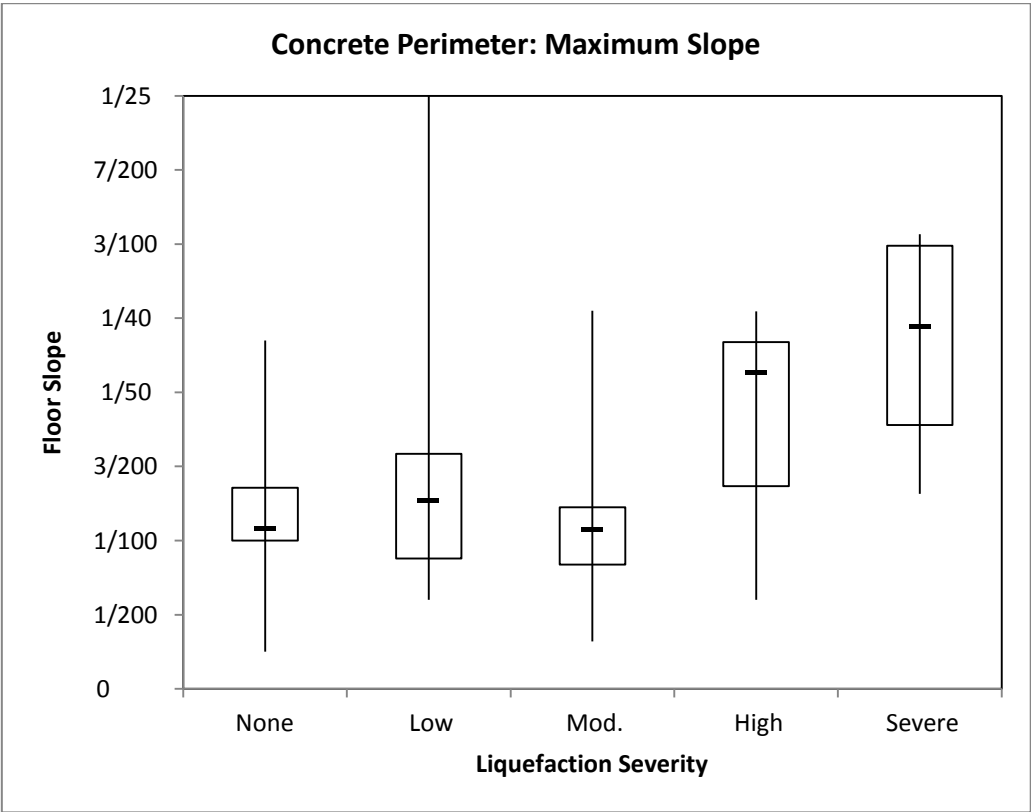


Figure A. 84. Box & Whisker plot for the maximum slope of concrete perimeter foundations for each level of liquefaction severity.

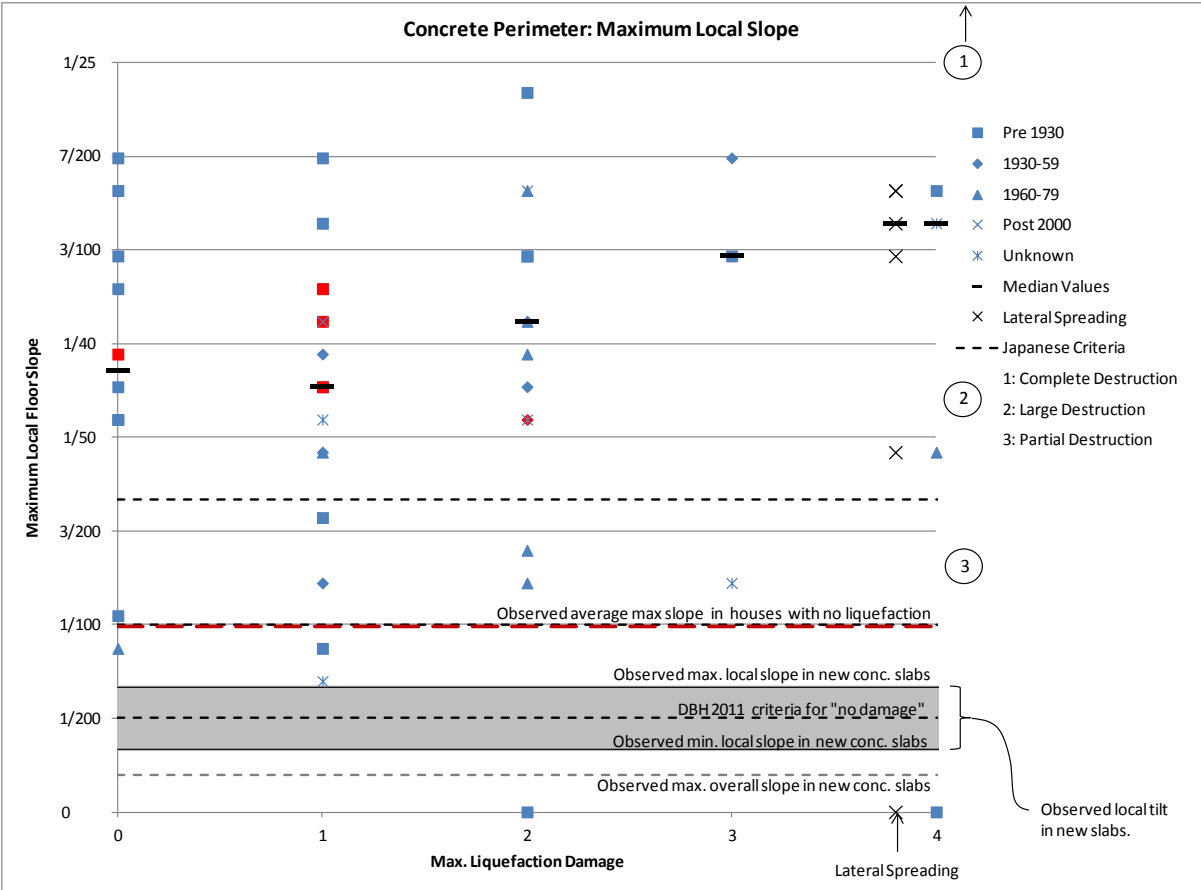


Figure A. 85. Maximum local floor slope vs. maximum liquefaction damage for concrete perimeter foundations, showing lateral spreading data and no damage references.

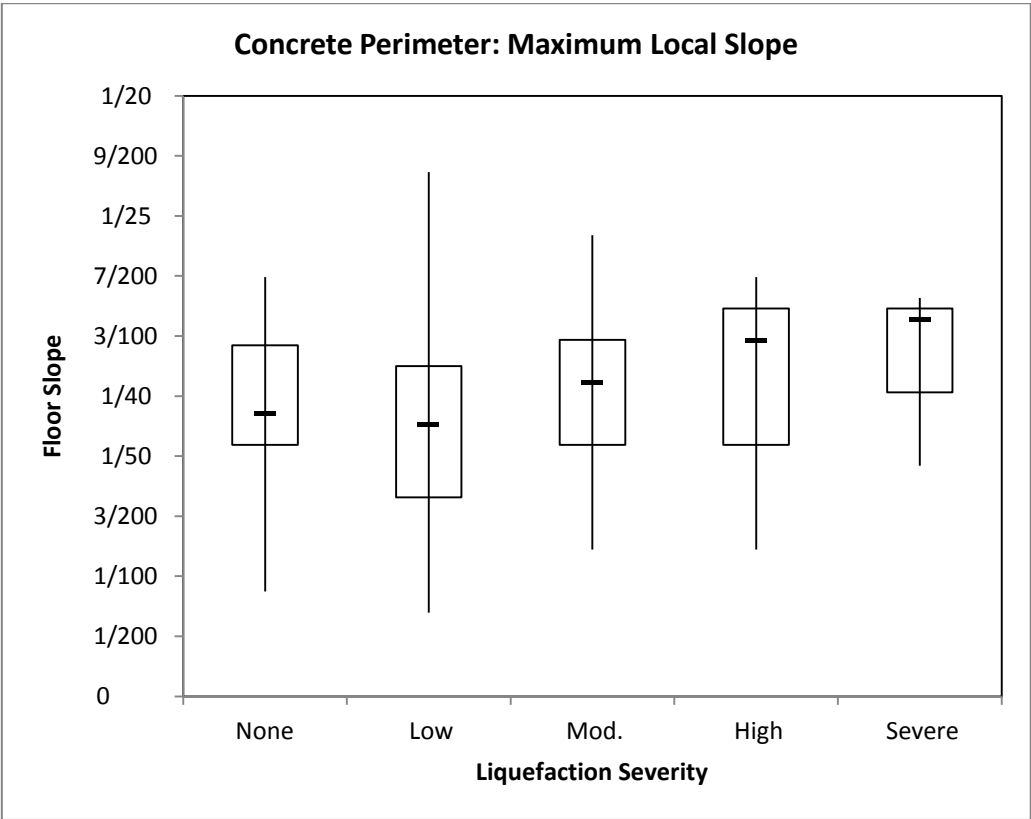


Figure A. 86. Box & Whisker plot for the maximum local slope of concrete perimeter foundations for each level of liquefaction severity.

N.1.2: Slab-on-Grade

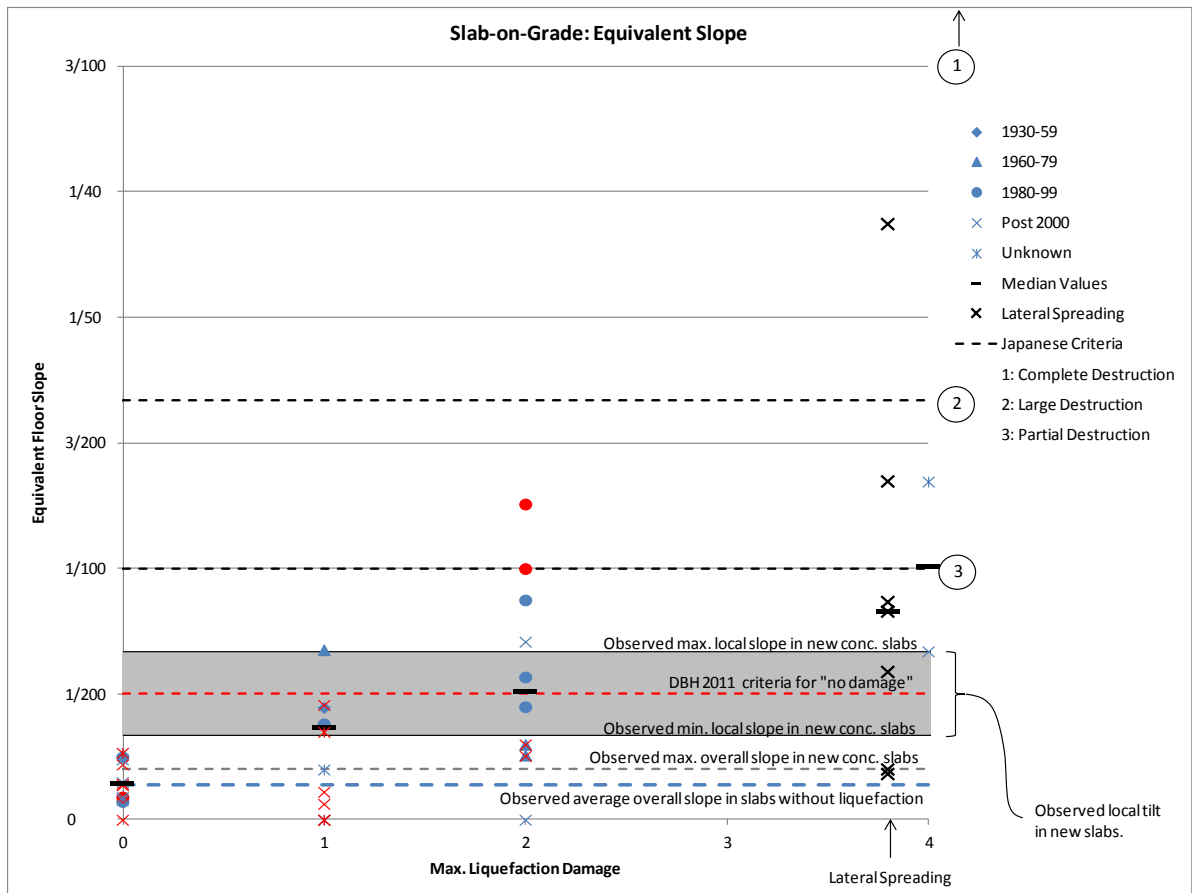


Figure A. 87. Equivalent floor slope vs. maximum liquefaction damage for slab-on-grade foundations, showing lateral spreading data and no damage references.

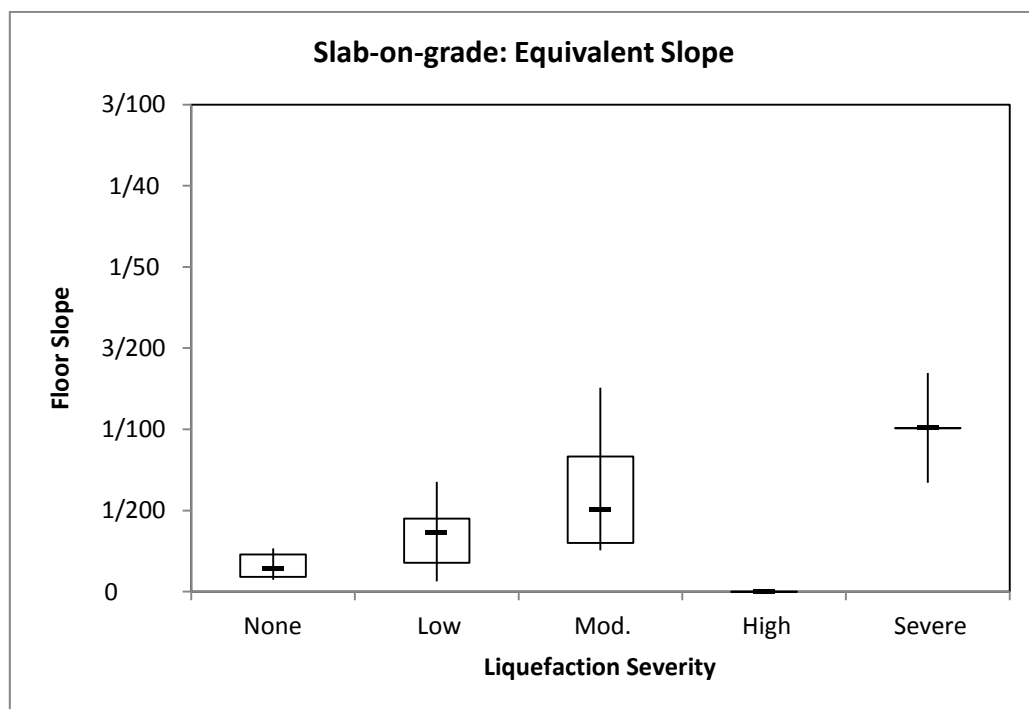


Figure A. 88. Box & Whisker plot for the equivalent slope of slab-on-grade foundations for each level of liquefaction severity.

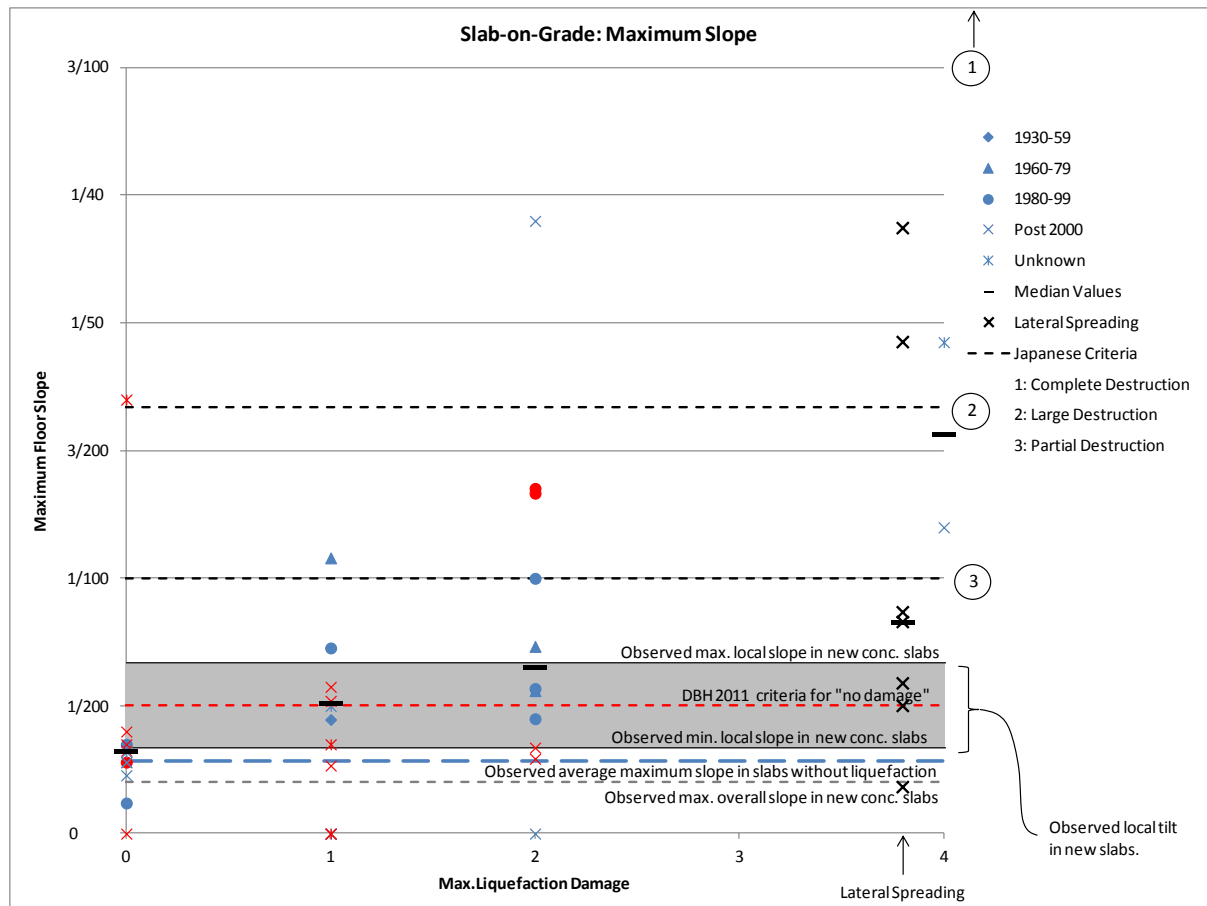


Figure A. 89. Maximum floor slope vs. maximum liquefaction damage for slab-on-grade foundations, showing lateral spreading data and no damage references.

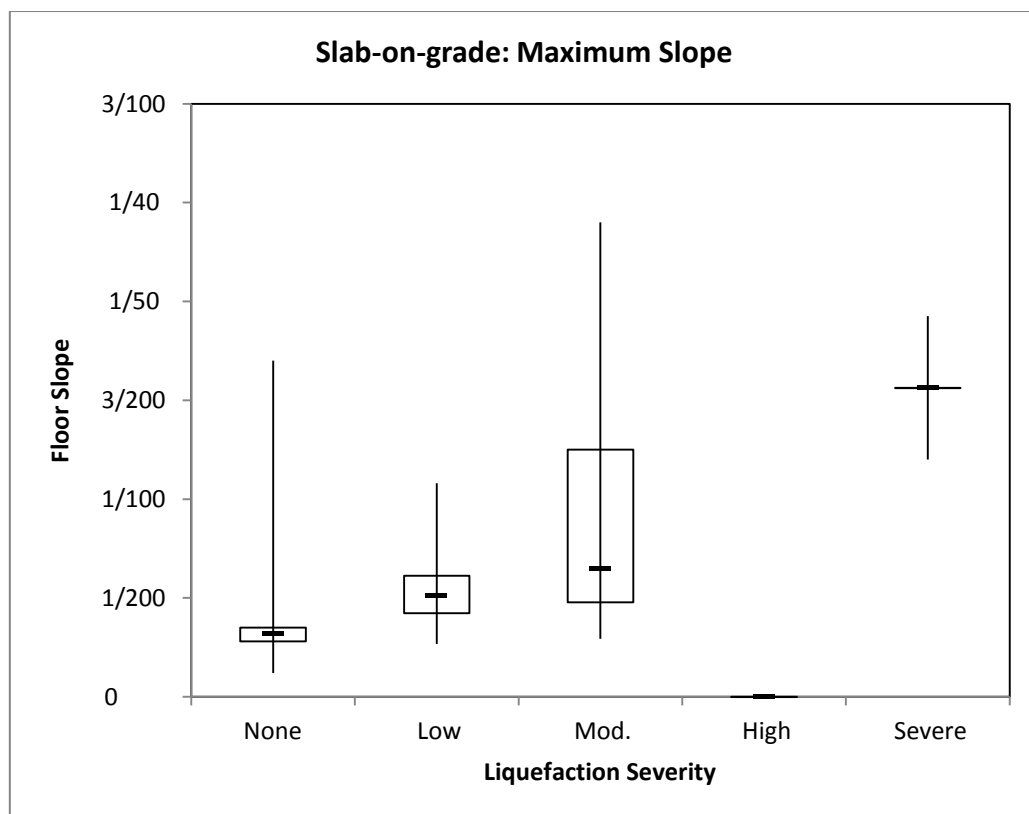


Figure A. 90. Box & Whisker plot for the maximum slope of slab-on-grade foundations for each level of liquefaction severity.

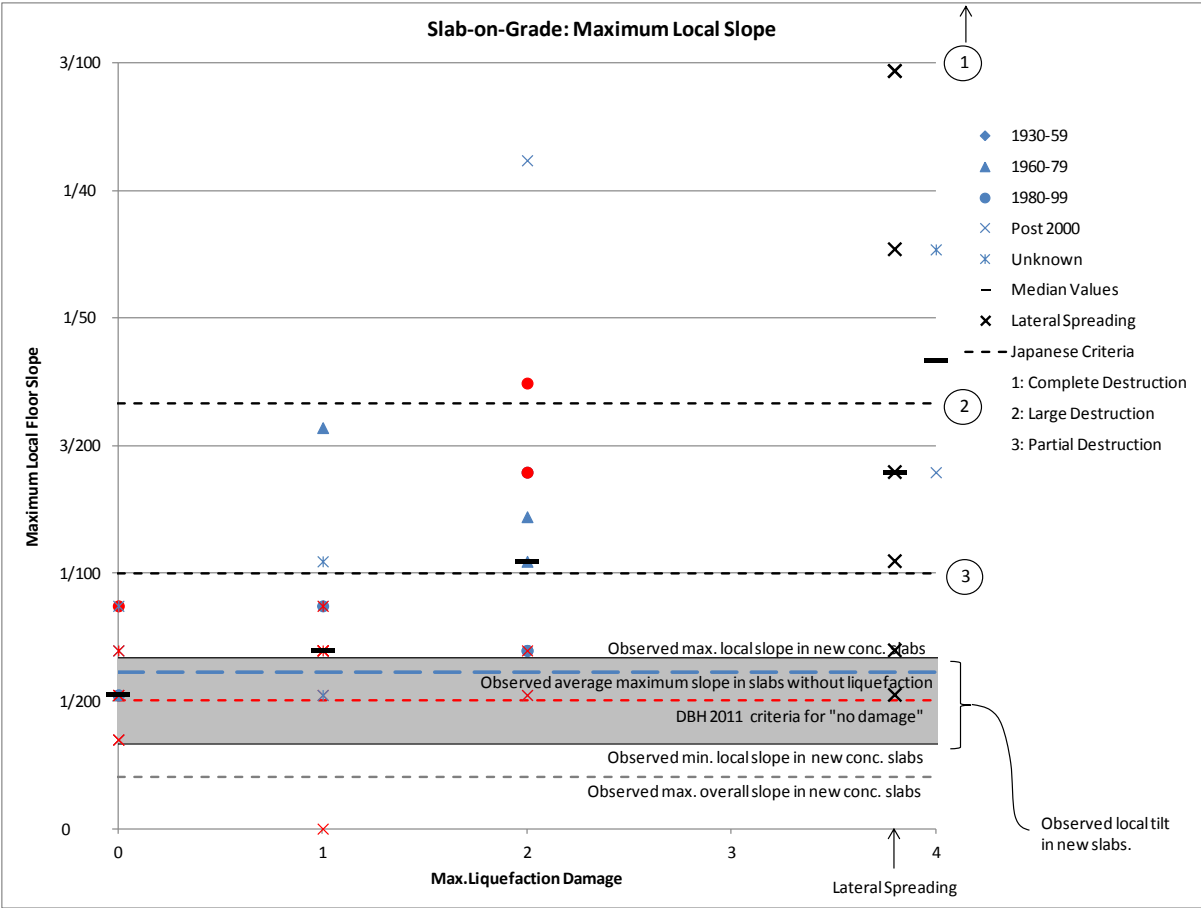


Figure A. 91. Maximum local floor slope vs. maximum liquefaction damage for slab-on-grade foundations, showing lateral spreading data and no damage references.

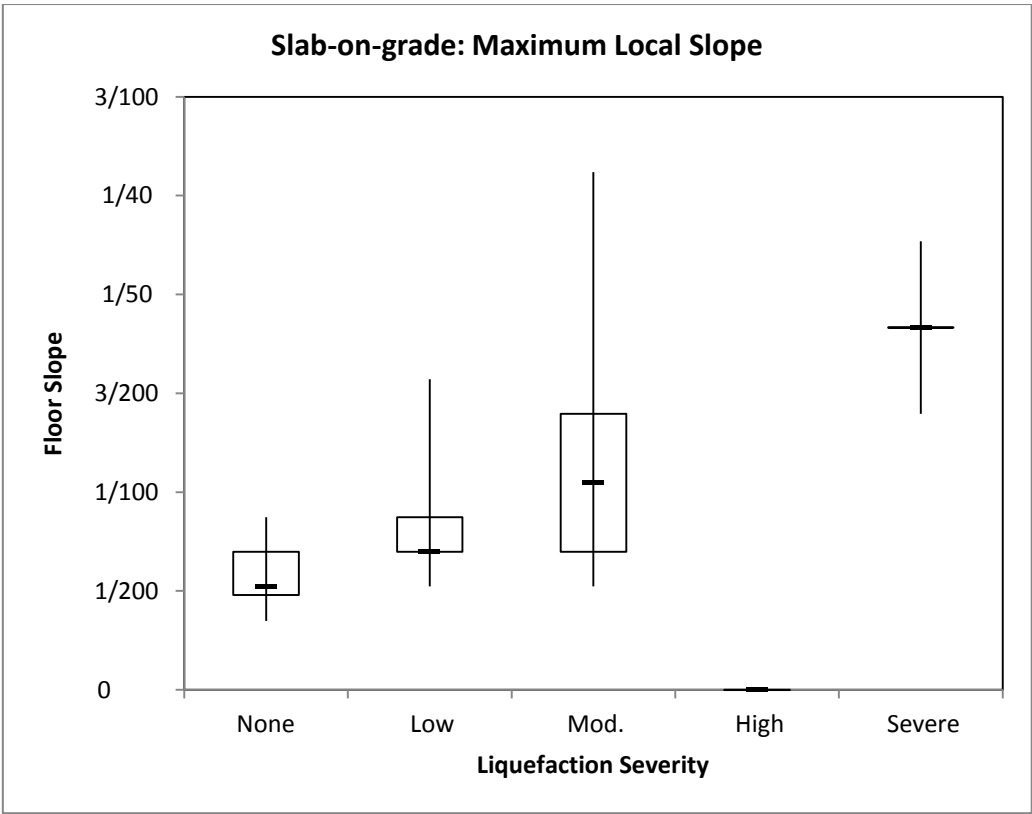


Figure A. 92. Box & Whisker plot for the maximum local slope of slab-on-grade foundations for each level of liquefaction severity.

N.1.3: RibRaft

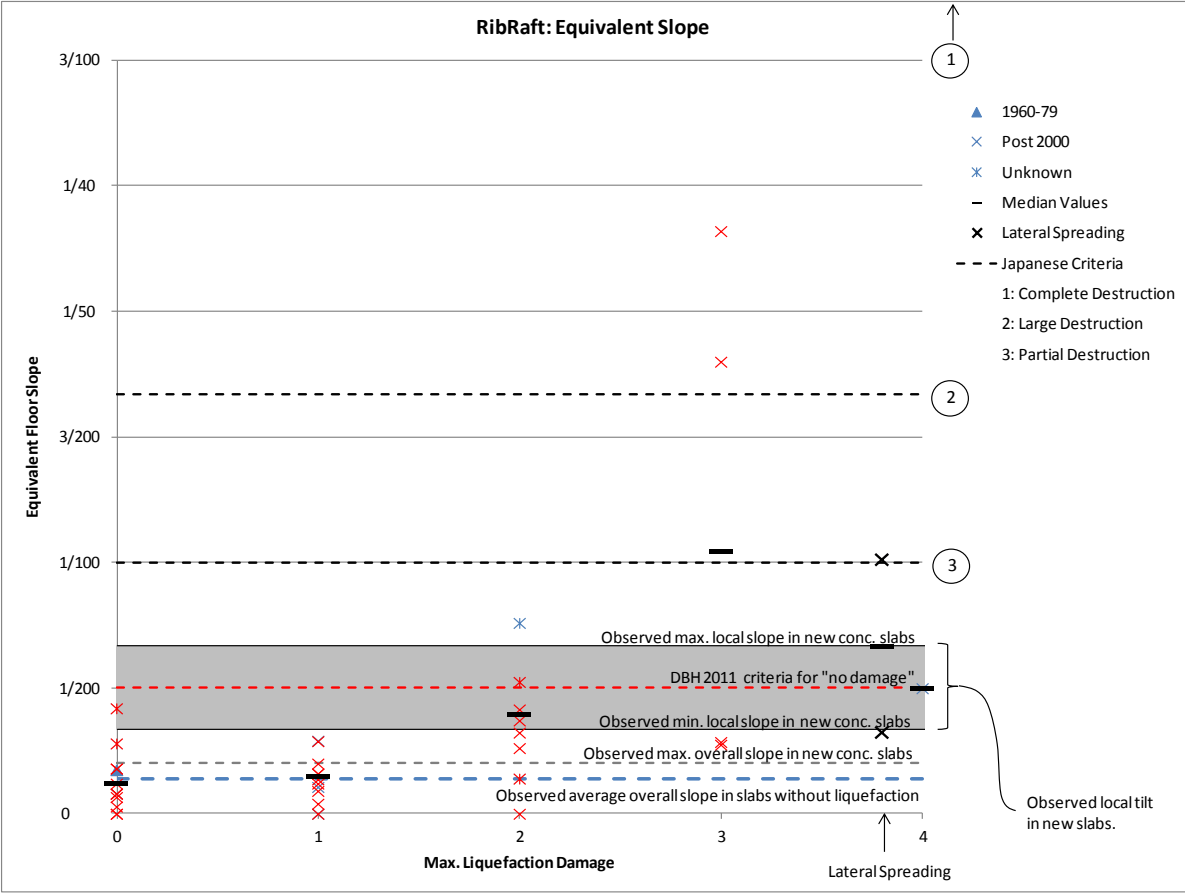


Figure A. 93. Equivalent floor slope vs. maximum liquefaction damage for ribraft foundations, showing lateral spreading data and no damage references.

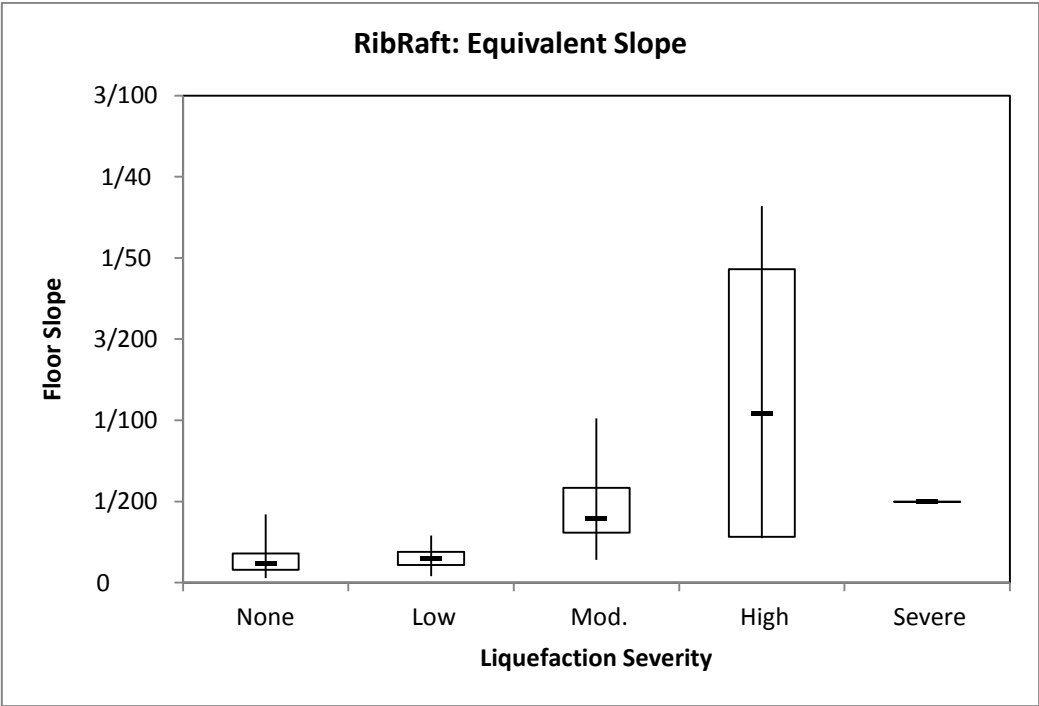


Figure A. 94. Box & Whisker plot for the equivalent slope of ribraft foundations for each level of liquefaction severity.

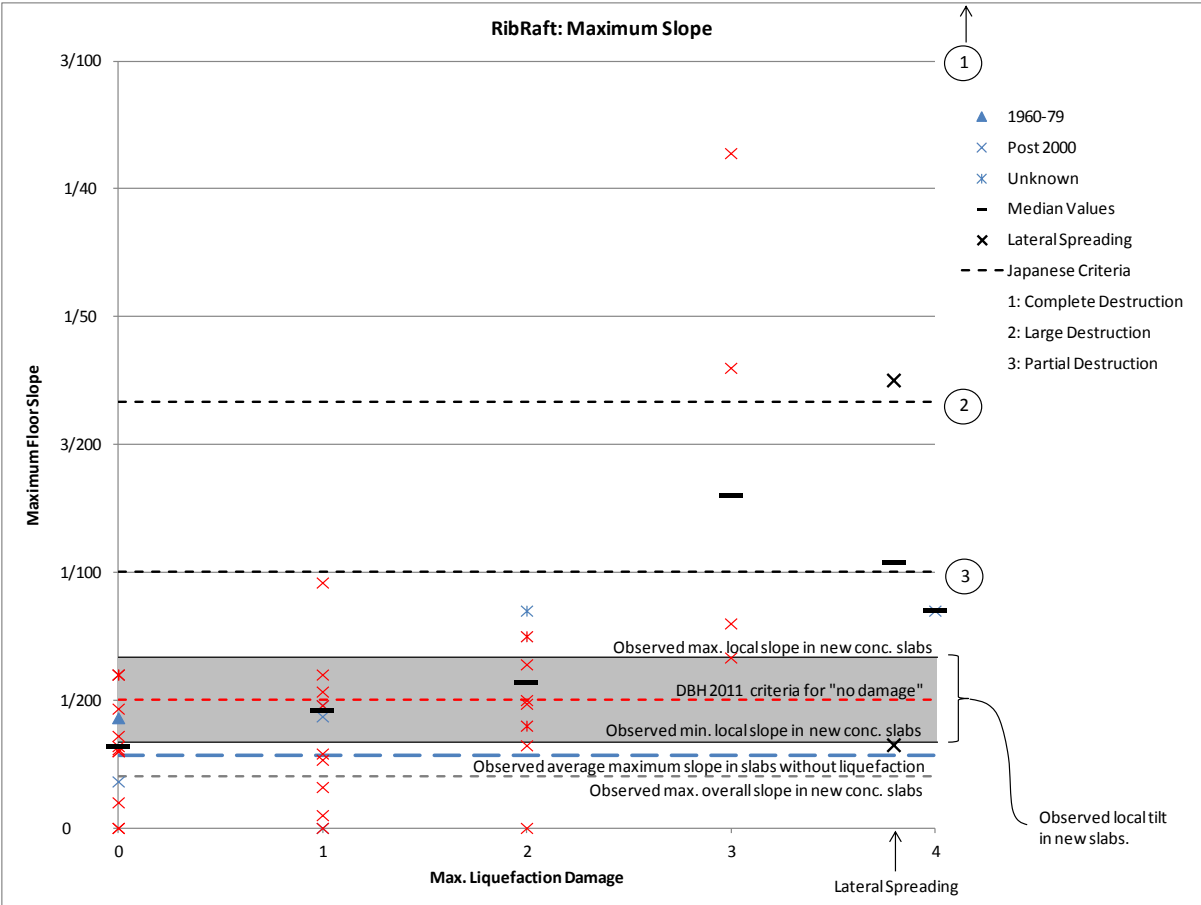


Figure A. 95. Maximum floor slope vs. maximum liquefaction damage for ribraft foundations, showing lateral spreading data and no damage references.

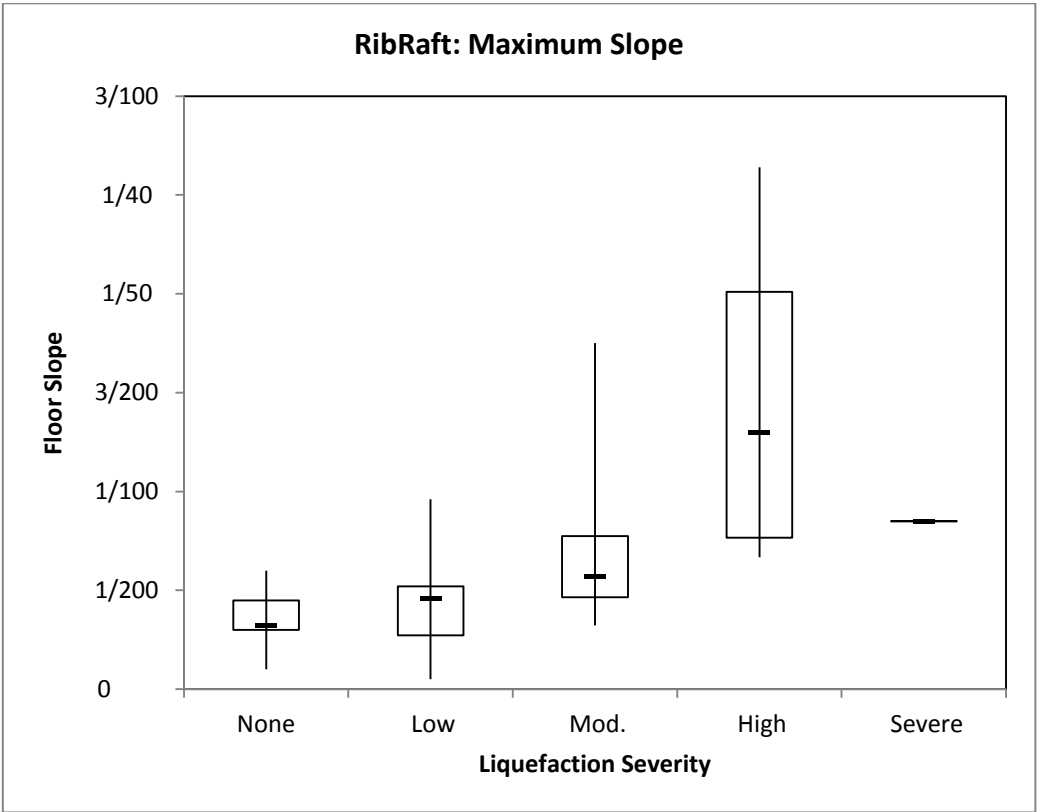


Figure A. 96. Box & Whisker plot for the maximum slope of ribraft foundations for each level of liquefaction severity.

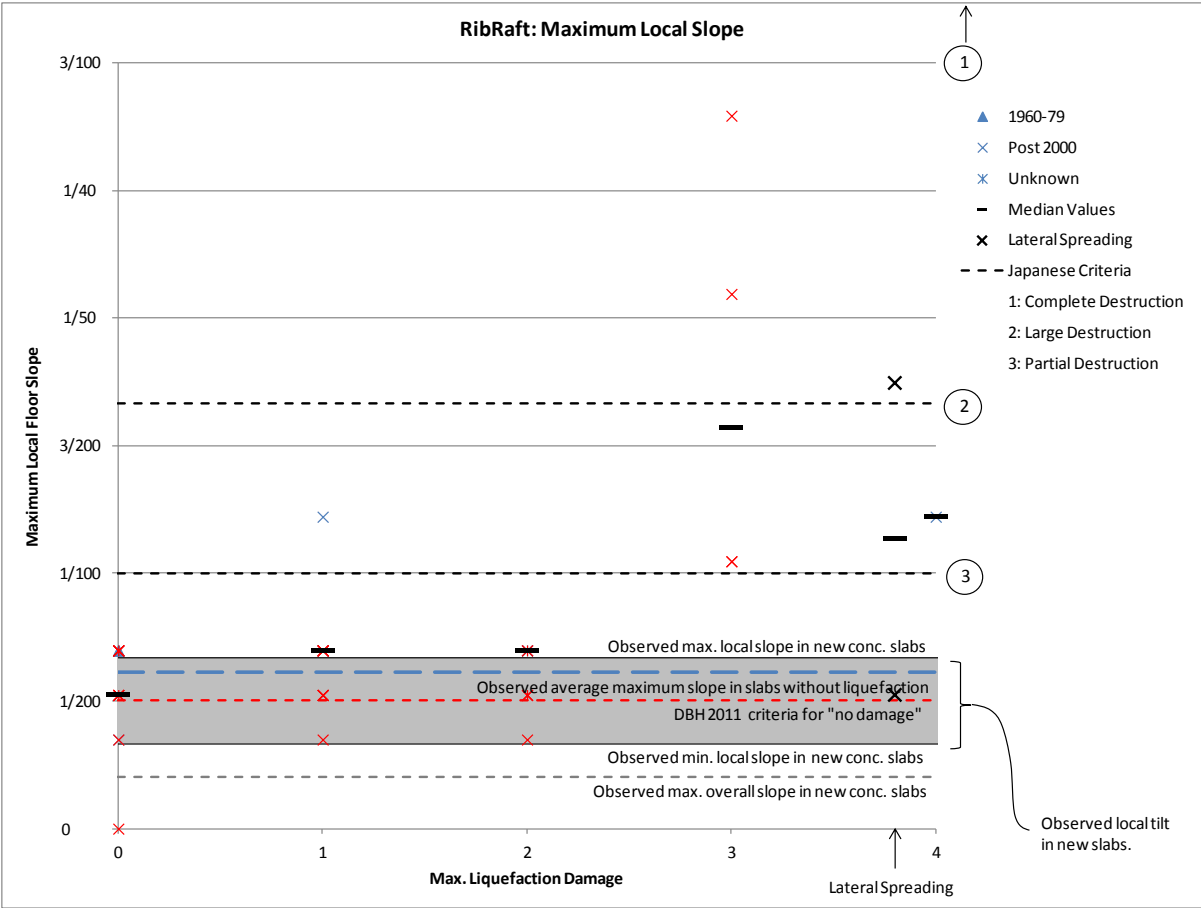


Figure A. 97. Maximum local floor slope vs. maximum liquefaction damage for ribraft foundations, showing lateral spreading data and no damage references.

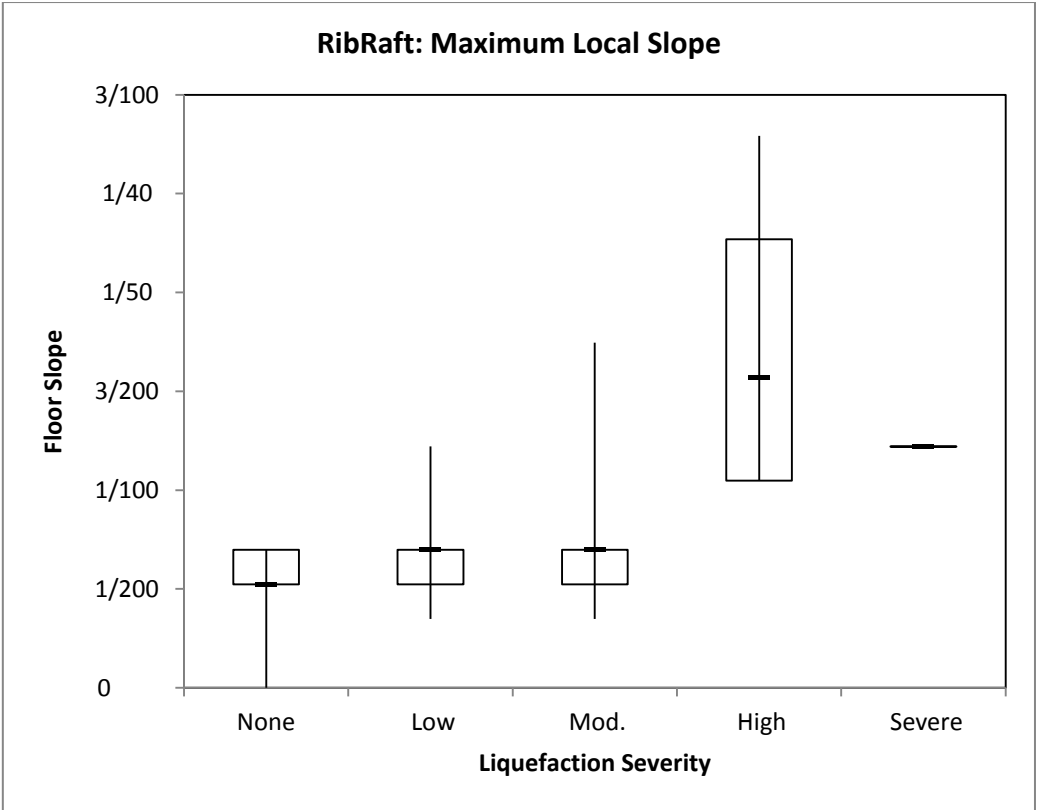


Figure A. 98. Box & Whisker plot for the maximum local slope of ribraft foundations for each level of liquefaction severity.

N.1.4: Piled

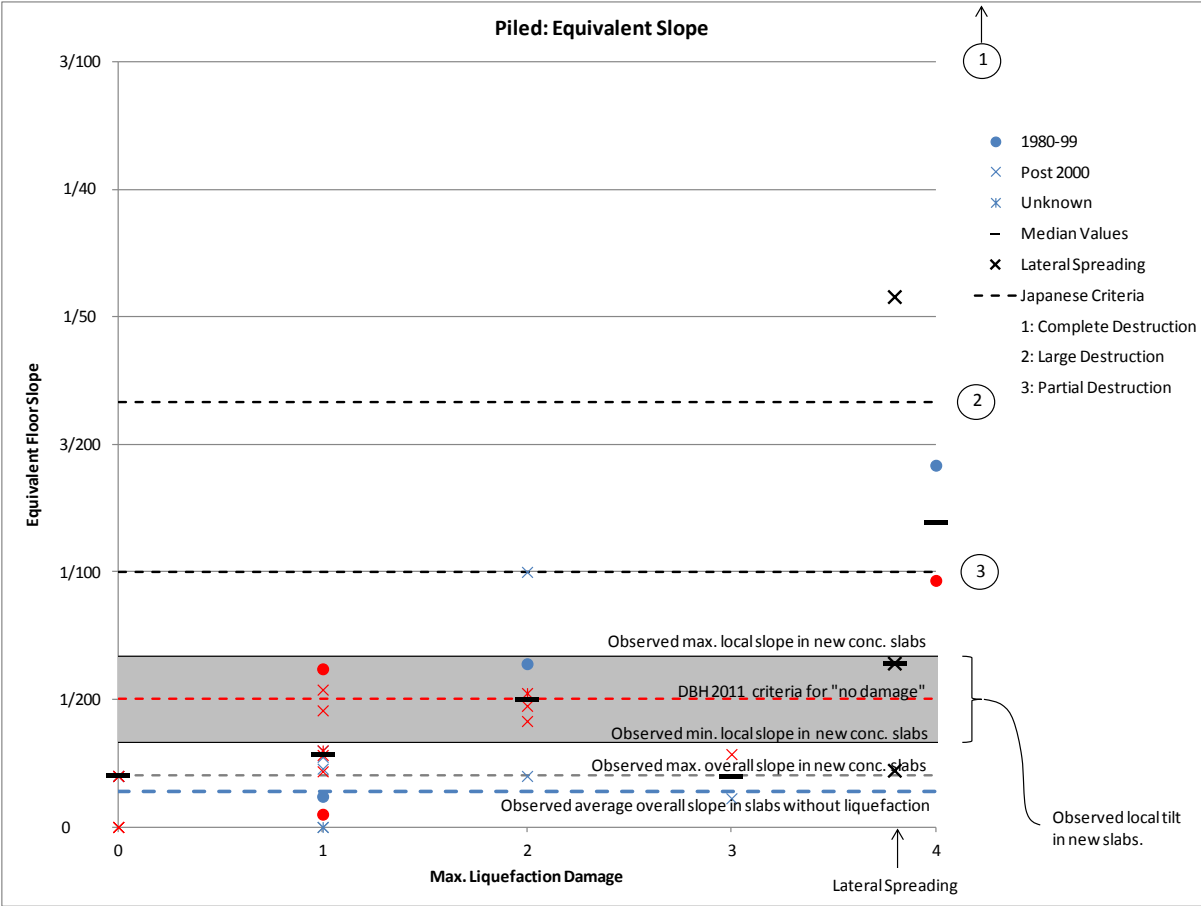


Figure A. 99. Equivalent floor slope vs. maximum liquefaction damage for piled foundations, showing lateral spreading data and no damage references.

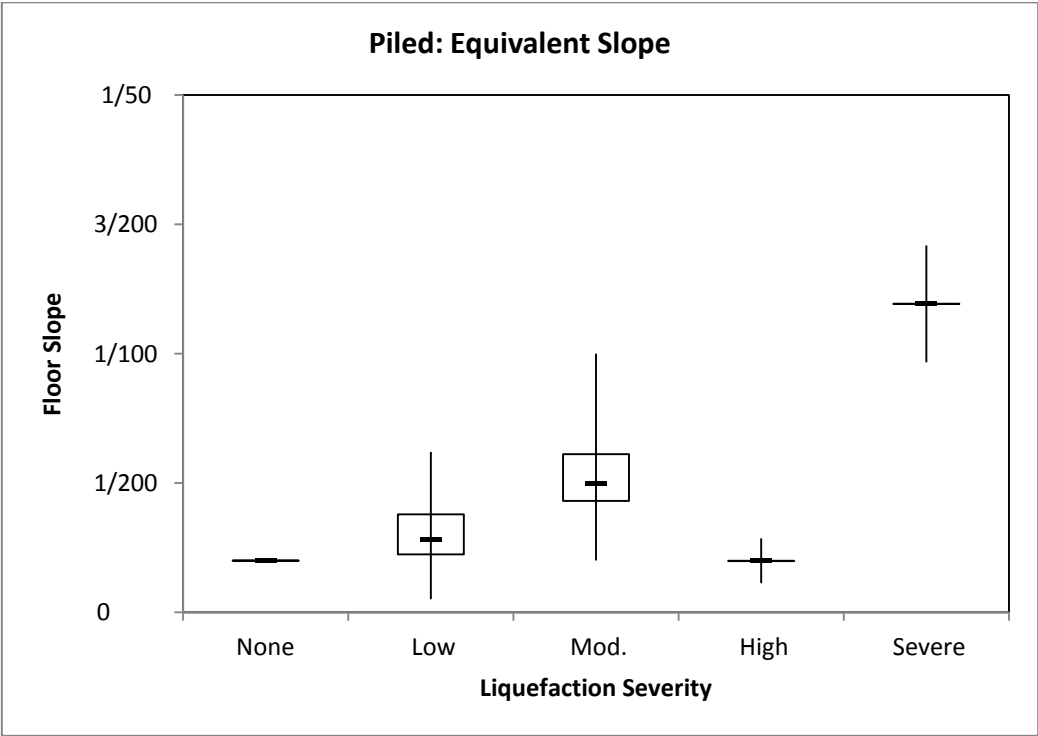


Figure A. 100. Box & Whisker plot for the equivalent slope of piled foundations for each level of liquefaction severity.

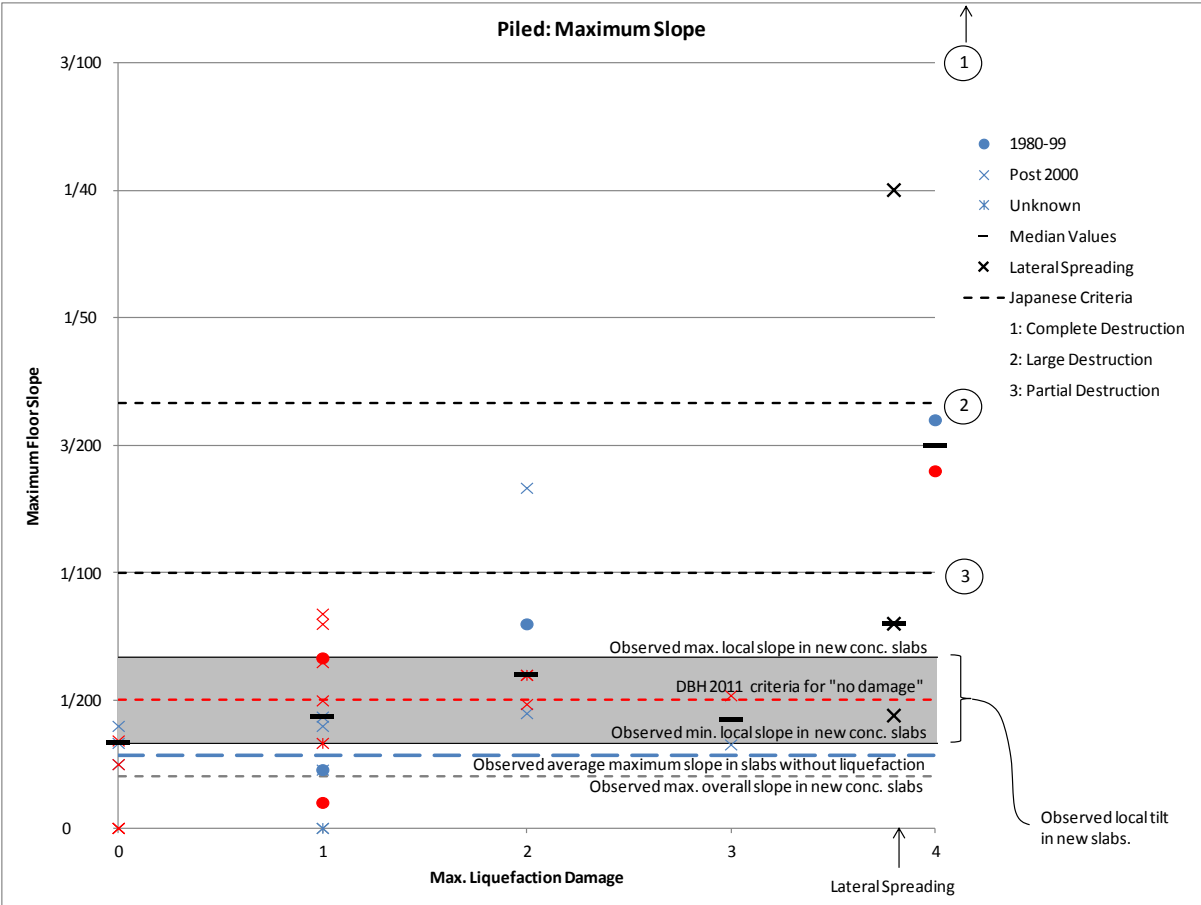


Figure A. 101. Maximum floor slope vs. maximum liquefaction damage for piled foundations, showing lateral spreading data and no damage references.

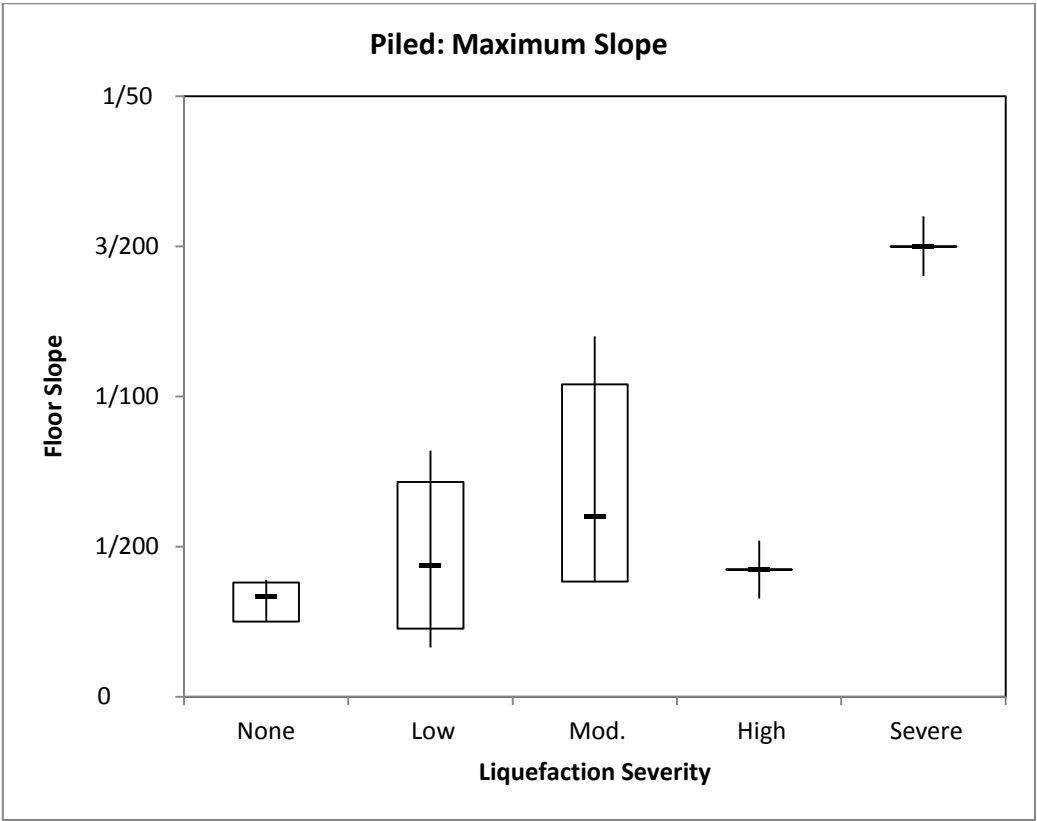


Figure A. 102. Box & Whisker plot for the maximum slope of piled foundations for each level of liquefaction severity.

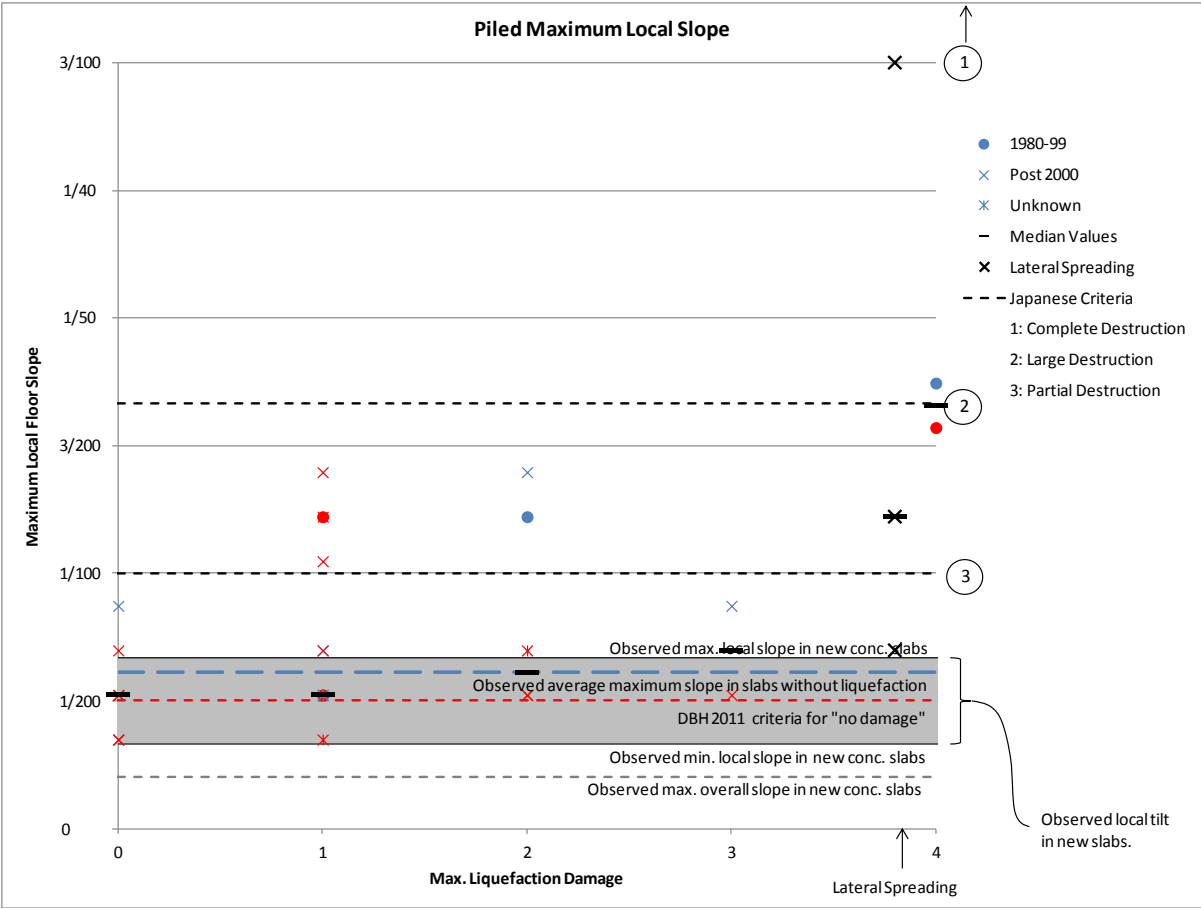


Figure A. 103. Maximum local floor slope vs. maximum liquefaction damage for piled foundations, showing lateral spreading data and no damage references.

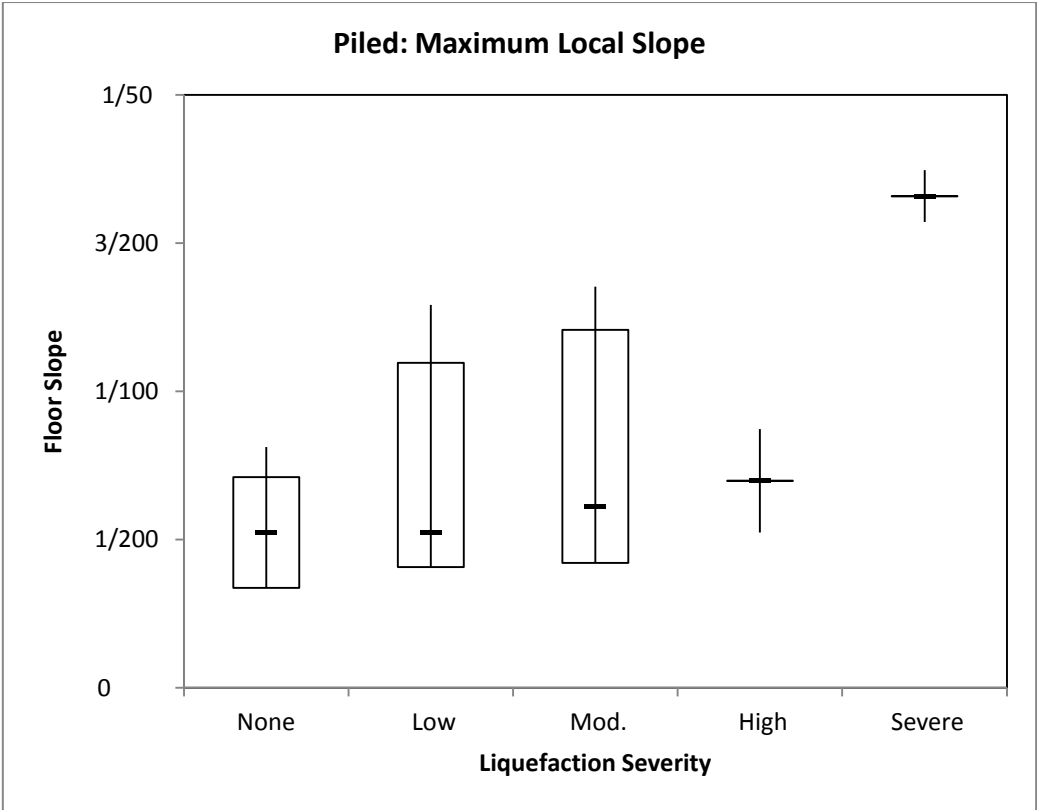


Figure A. 104. Box & Whisker plot for the maximum local slope of piled foundations for each level of liquefaction severity.

N.1.5: Median Comparisons

N.1.5.1: By Foundation Type

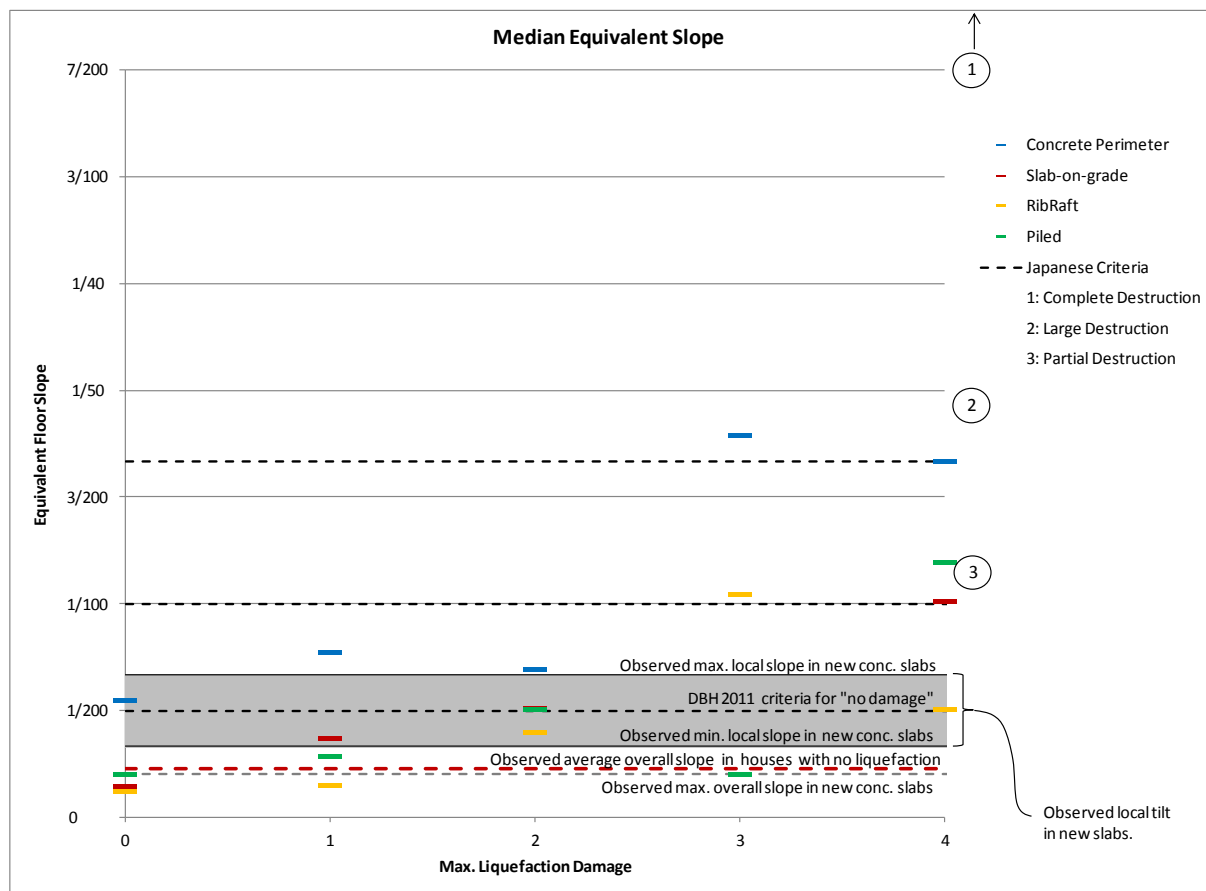


Figure A. 105. Median equivalent floor slopes at each liquefaction severity level for all foundation types.

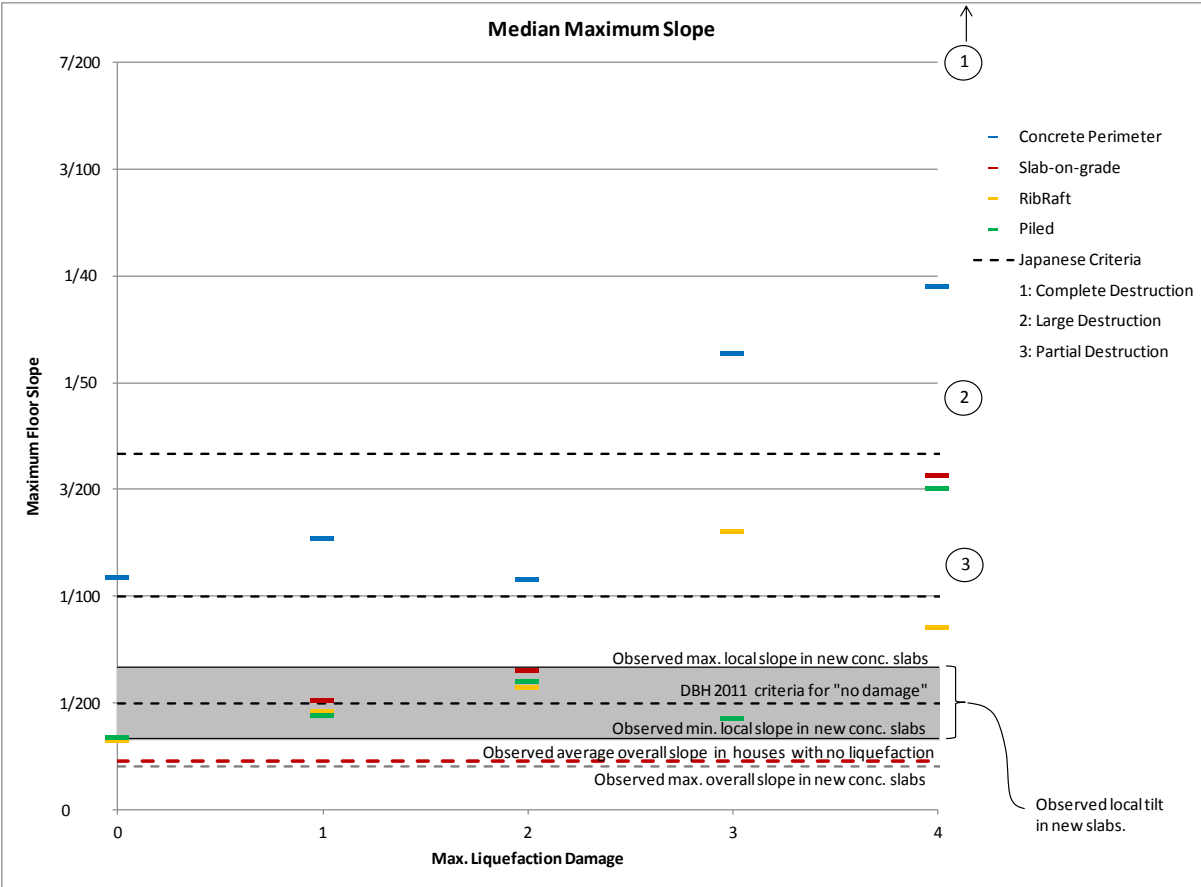


Figure A. 106. Median maximum floor slopes at each liquefaction severity level for all foundation types.

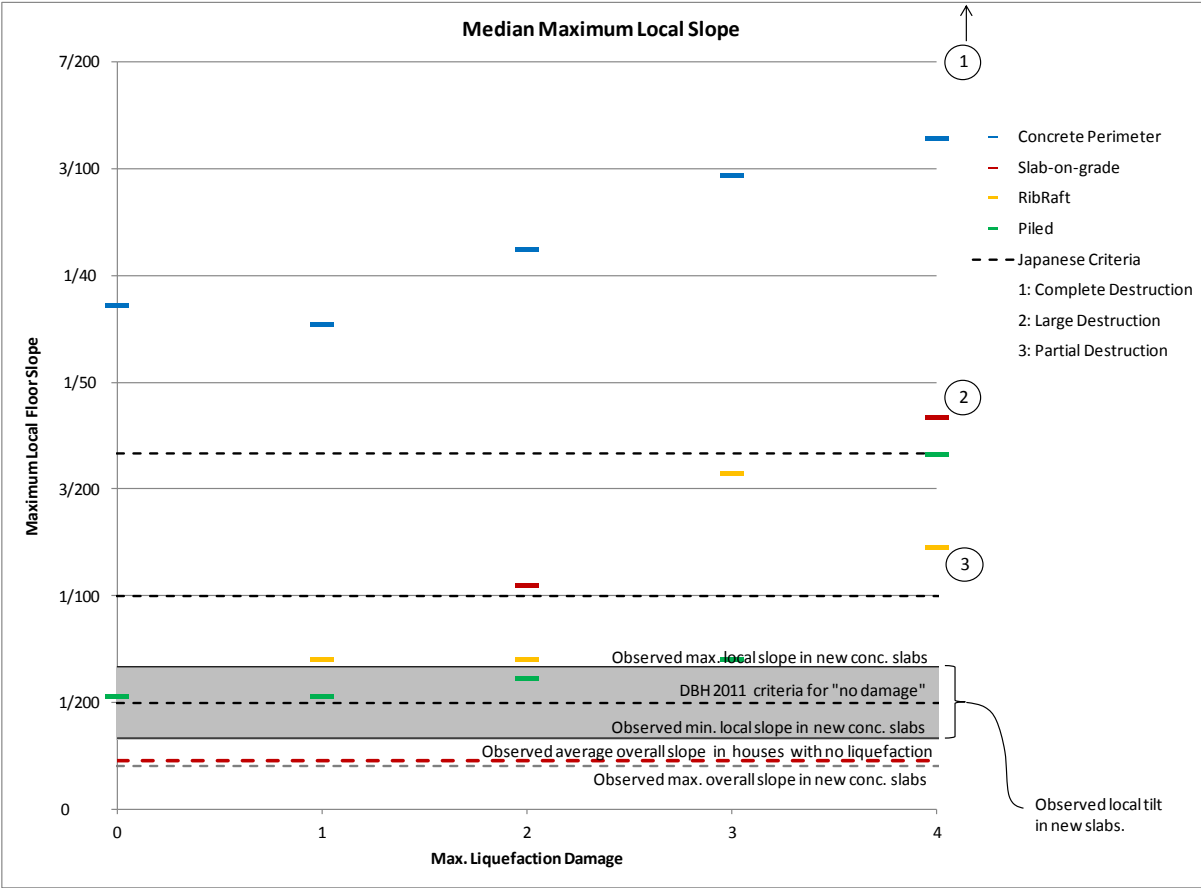


Figure A. 107. Median maximum local floor slopes at each liquefaction severity level for all foundation types.

N.1.5.2: By Liquefaction Severity

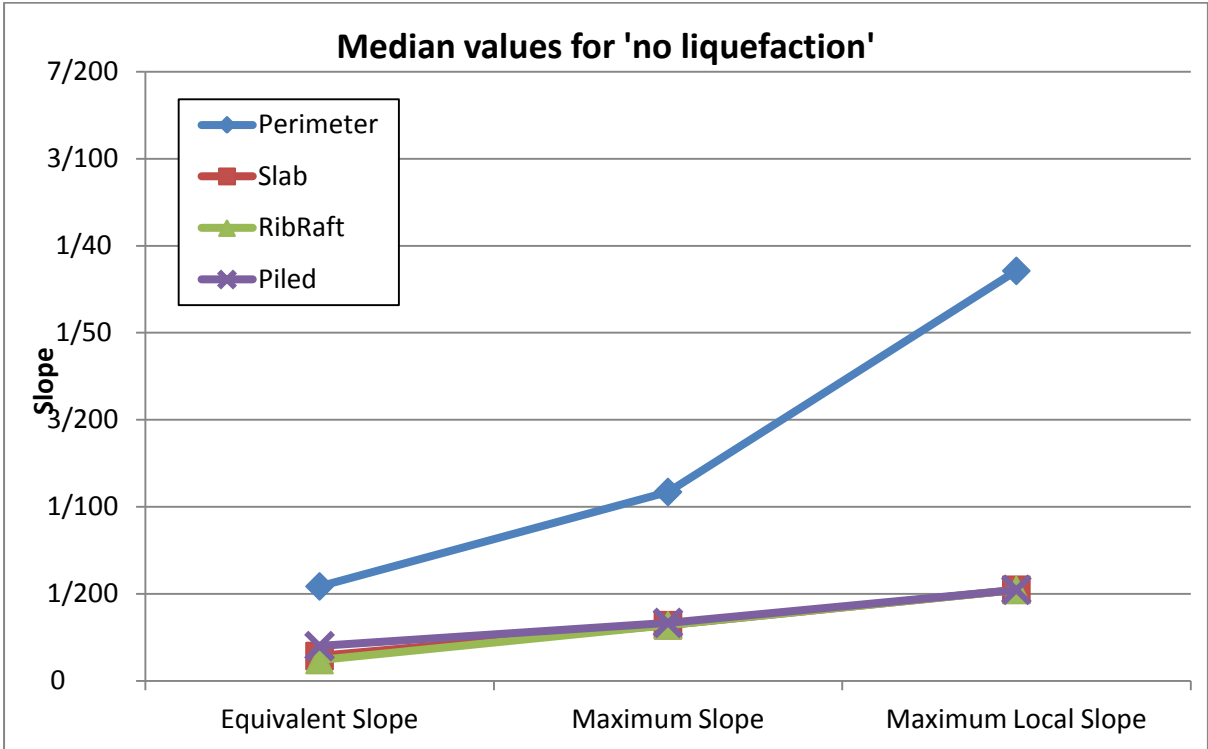


Figure A. 108. Median floor slope values for all floor slope indices and foundations, for no liquefaction.

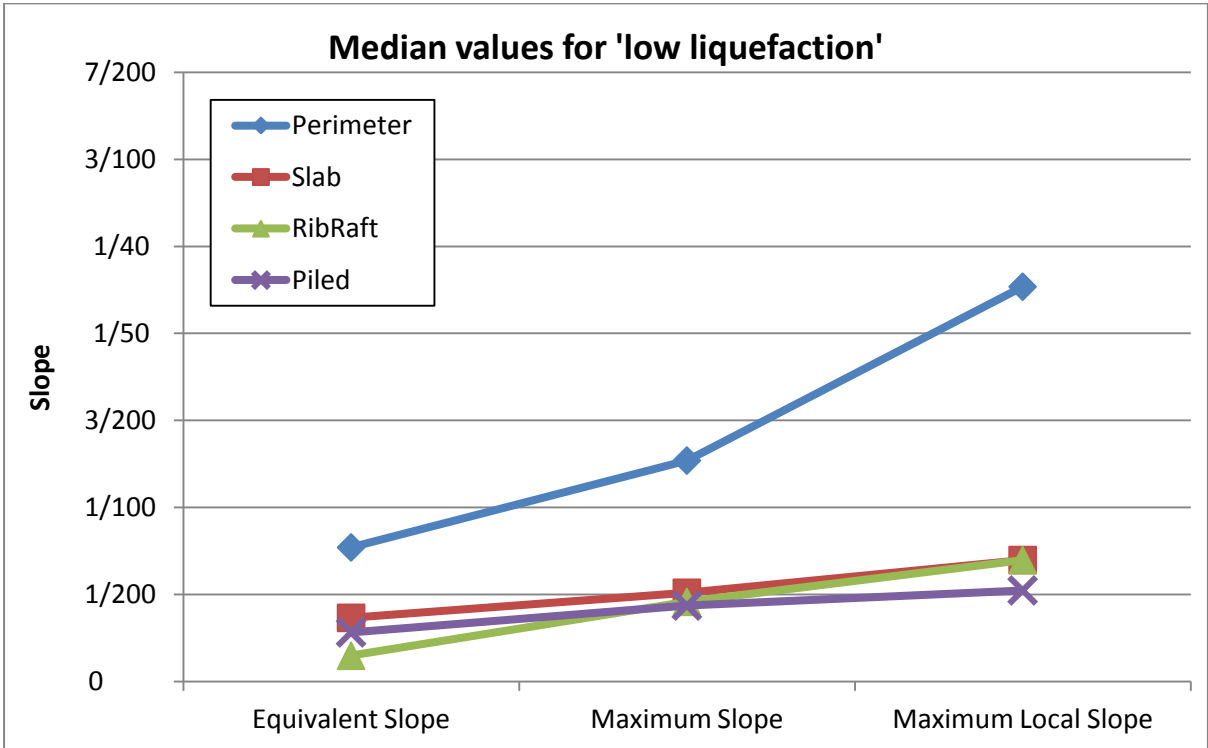


Figure A. 109. Median floor slope values for all floor slope indices and foundations, for low liquefaction.

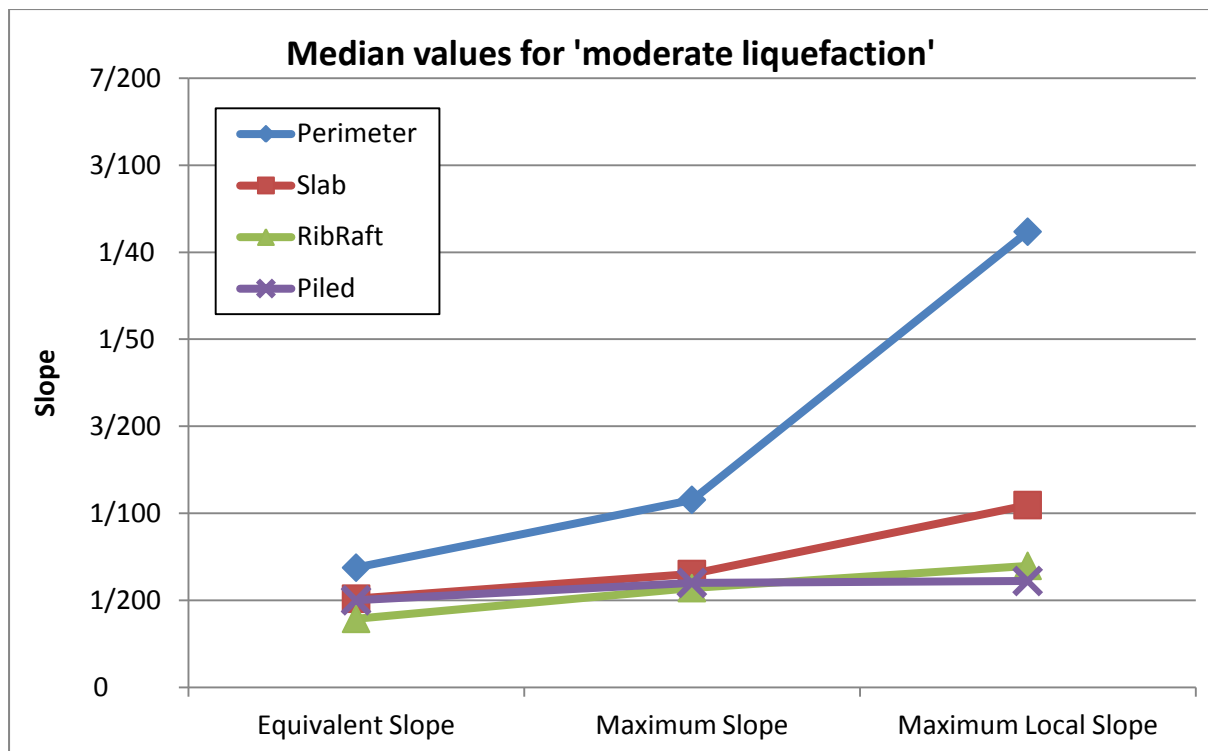


Figure A. 110. Median floor slope values for all floor slope indices and foundations, for moderate liquefaction.

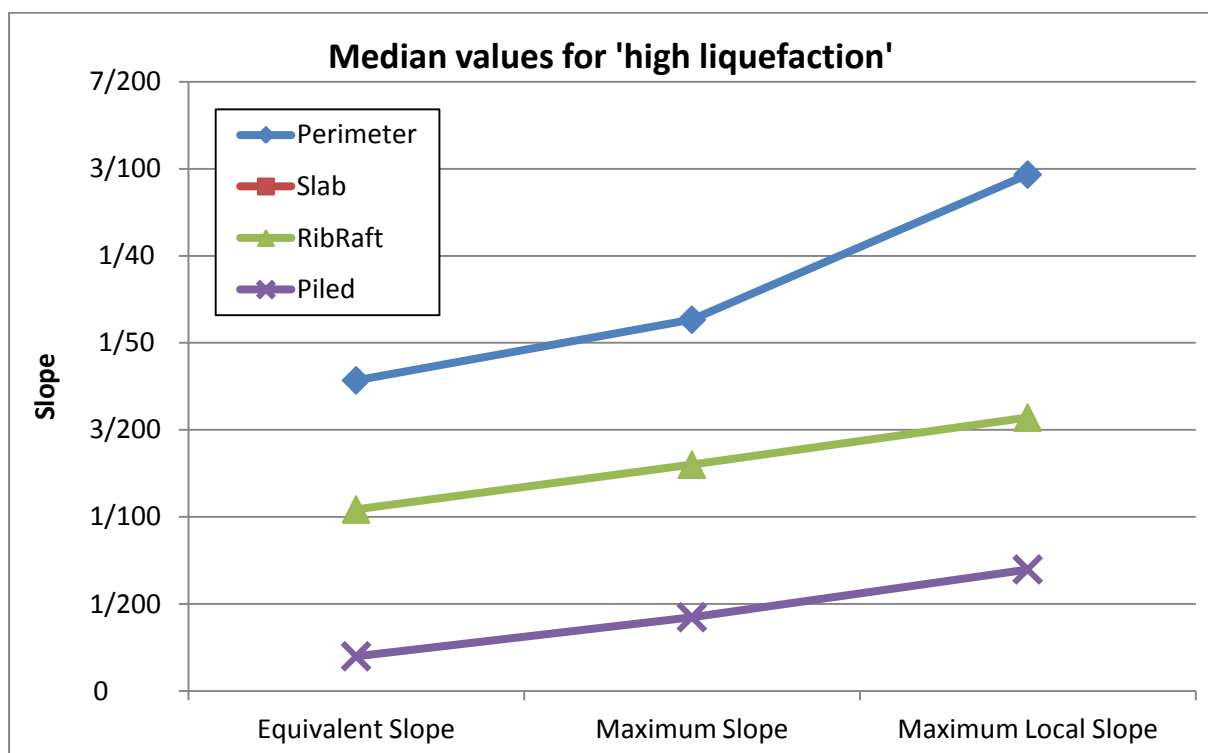


Figure A. 111. Median floor slope values for all floor slope indices and foundations, for high liquefaction.

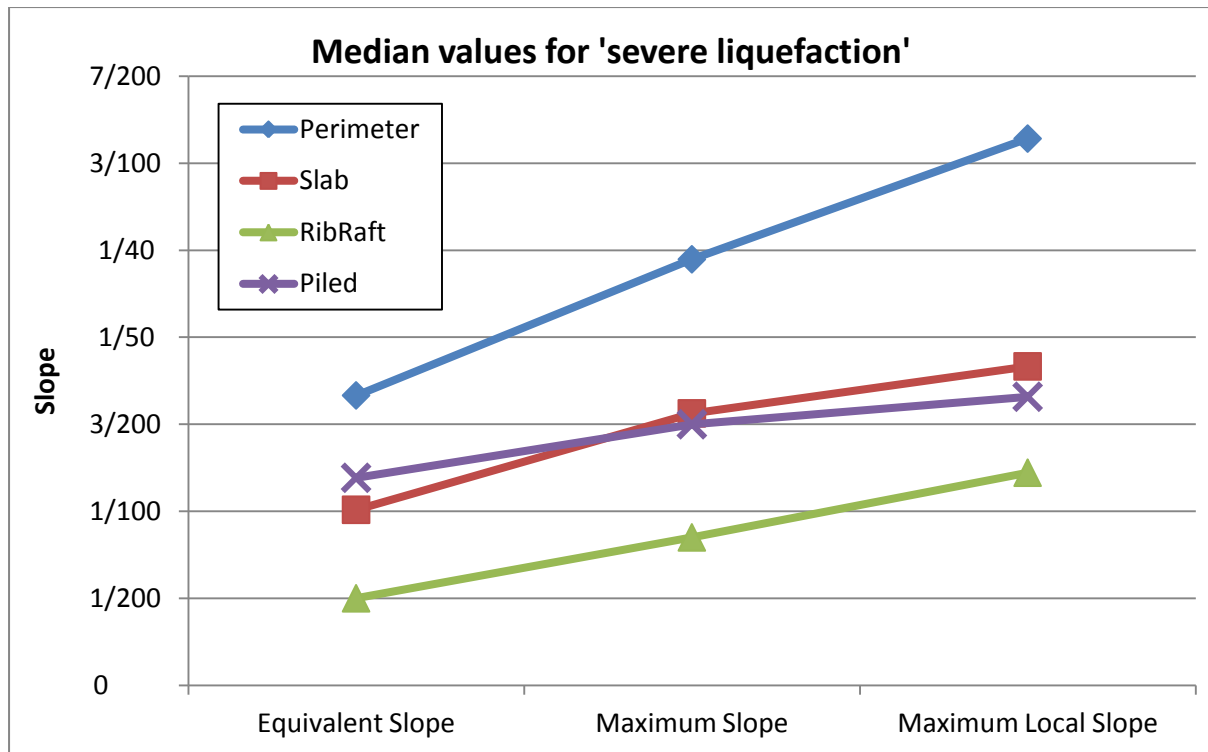


Figure A. 112. Median floor slope values for all floor slope indices and foundations, for severe liquefaction.

N.2: LiDAR Ground Settlement vs. Foundation Damage Parameters

N.2.1: Concrete Perimeter

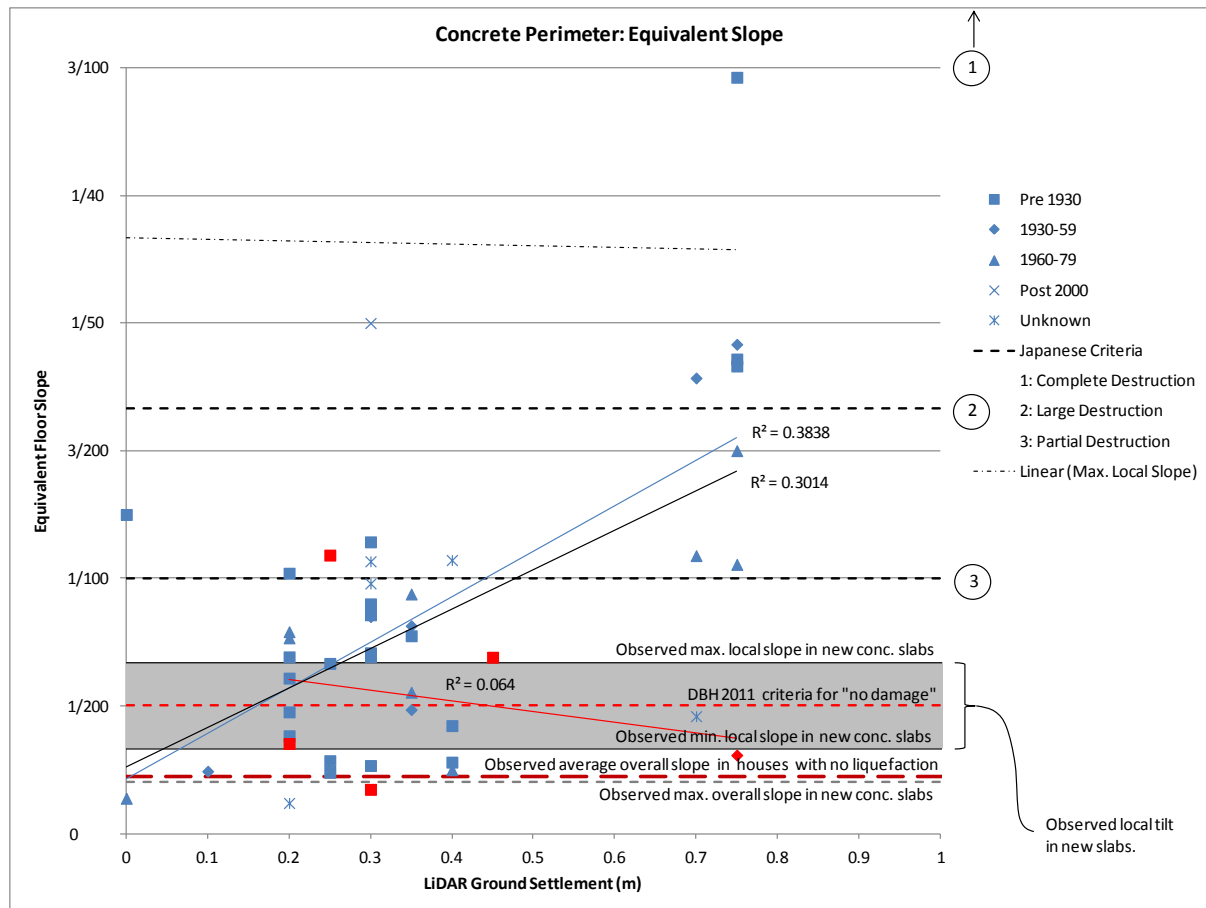


Figure A. 113. Equivalent floor slope vs. LiDAR ground settlement for concrete perimeter foundations, showing no damage references.

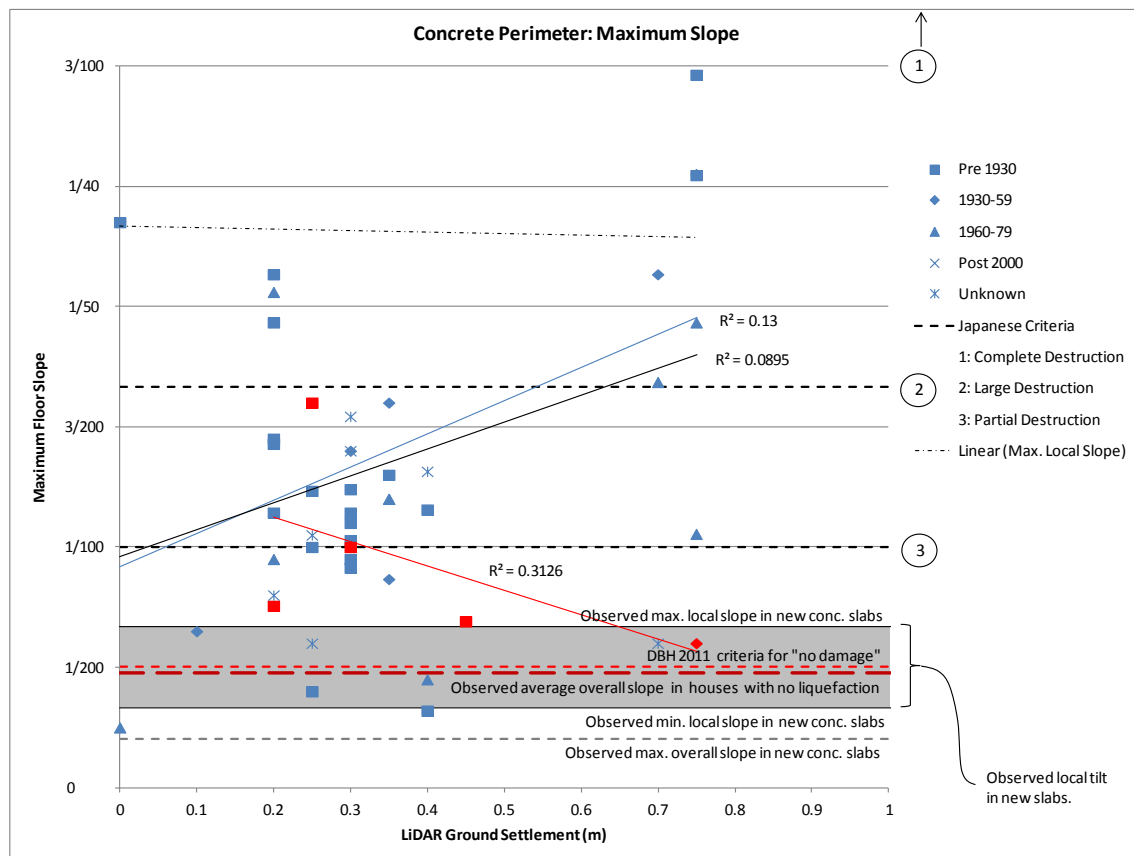


Figure A. 114. Maximum floor slope vs. LiDAR ground settlement for concrete perimeter foundations, showing no damage references.

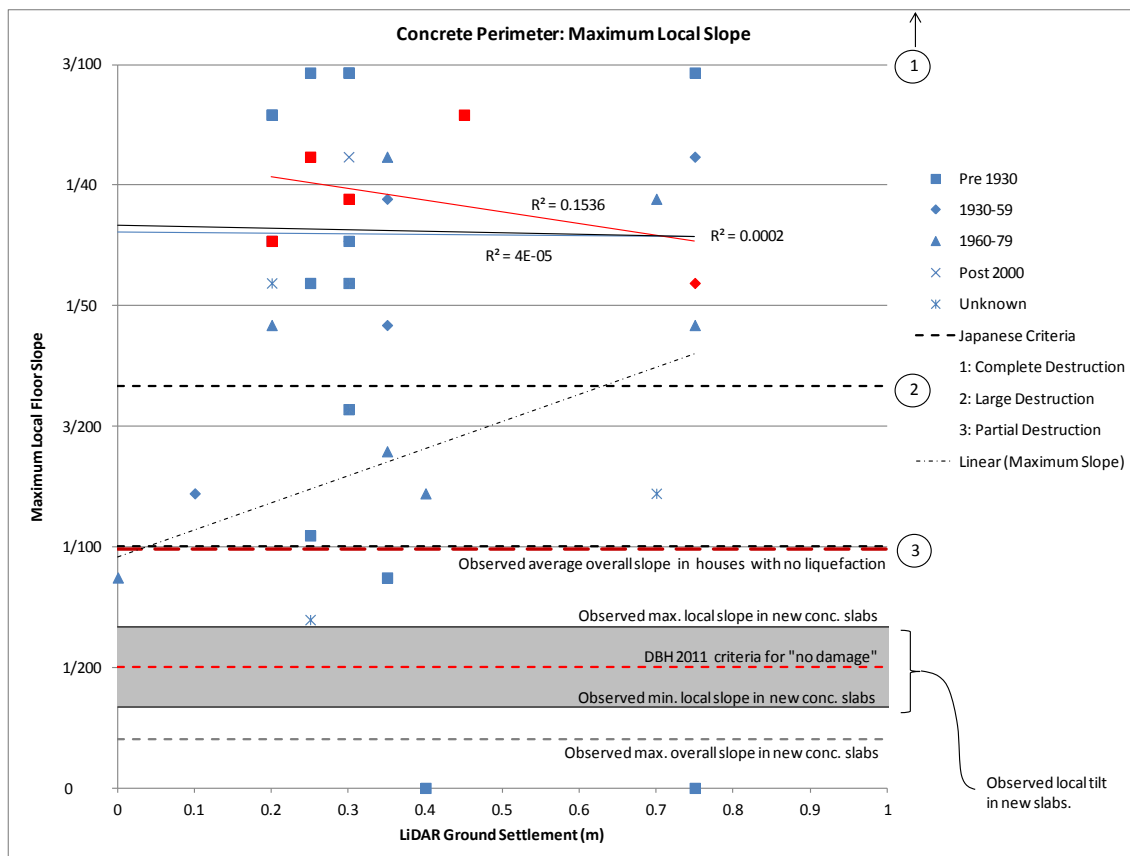


Figure A. 115. Maximum local floor slope vs. LiDAR ground settlement for concrete perimeter foundations, showing no damage references.

N.2.2: Slab-on-Grade

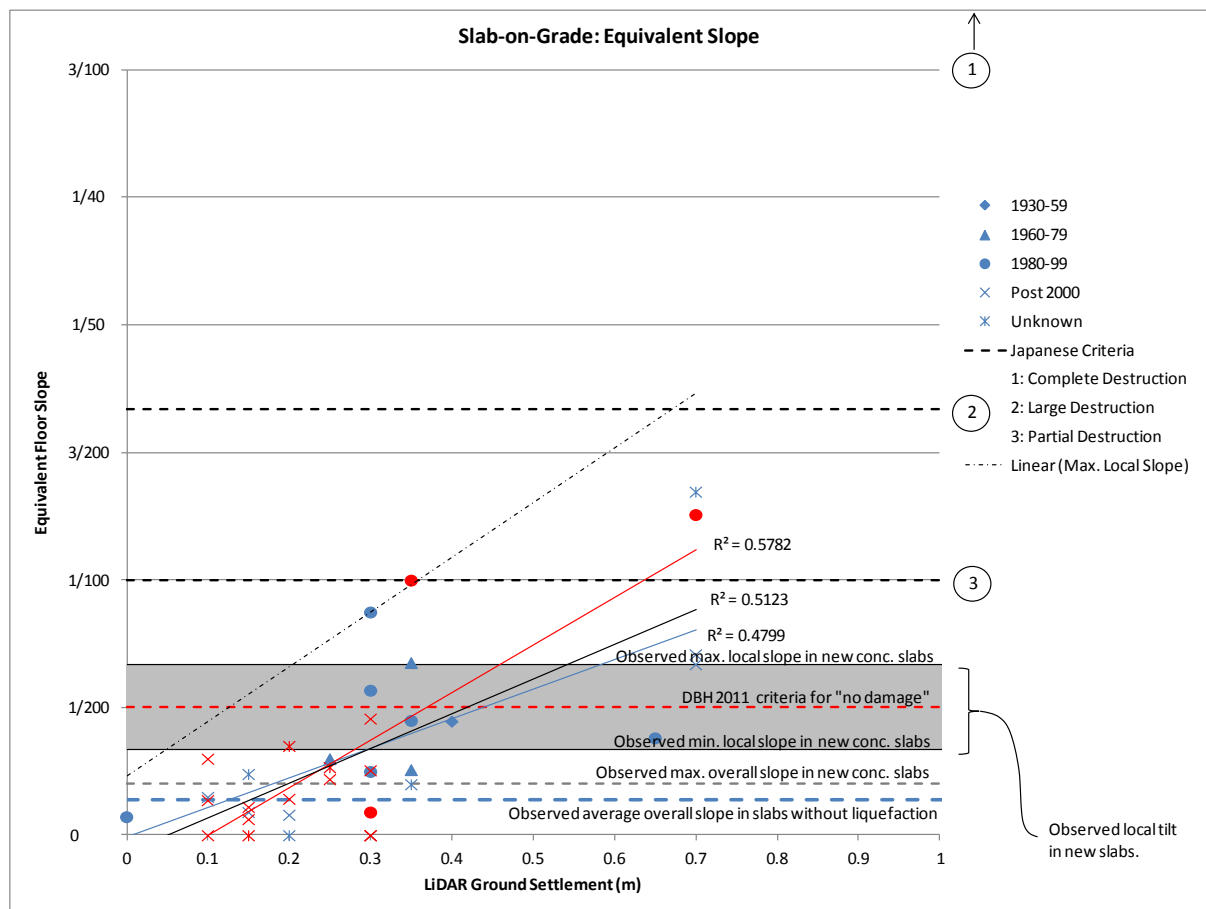


Figure A. 116. Equivalent floor slope vs. LiDAR ground settlement for slab-on-grade foundations, showing no damage references.

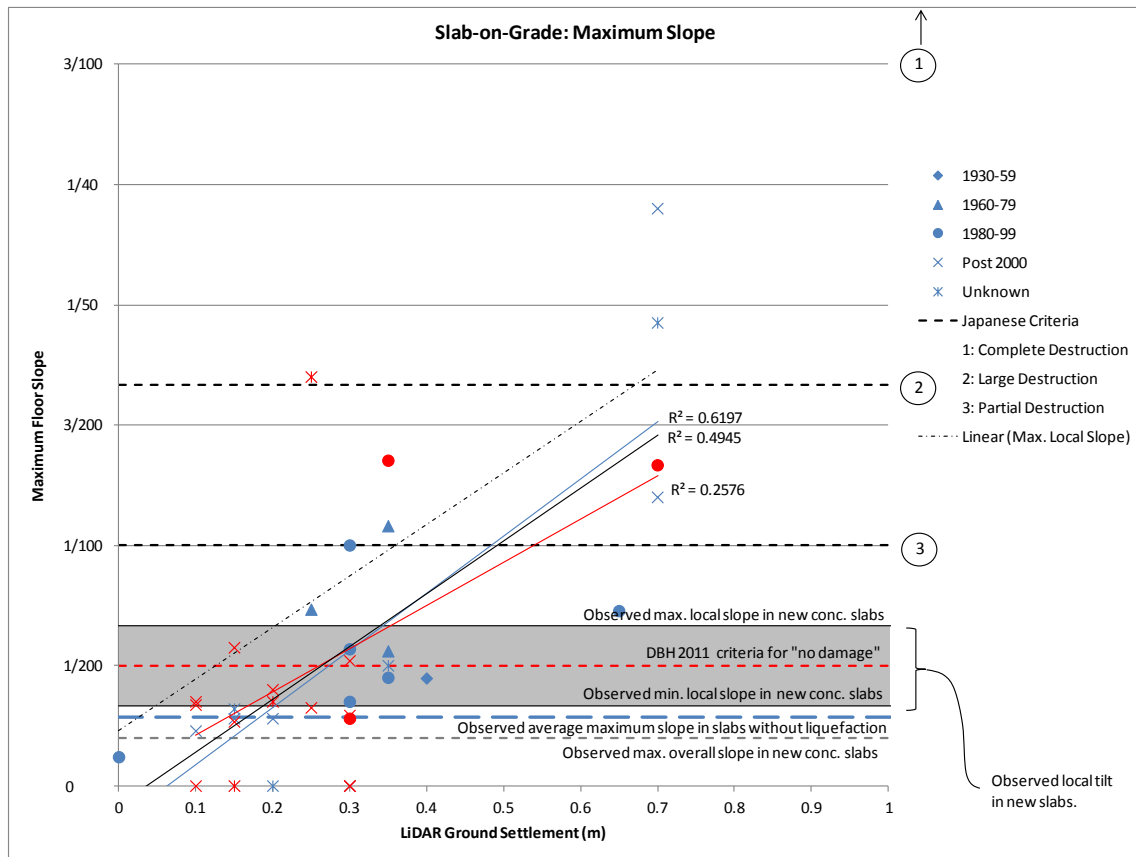


Figure A. 117. Maximum floor slope vs. LiDAR ground settlement for slab-on-grade foundations, showing no damage references.

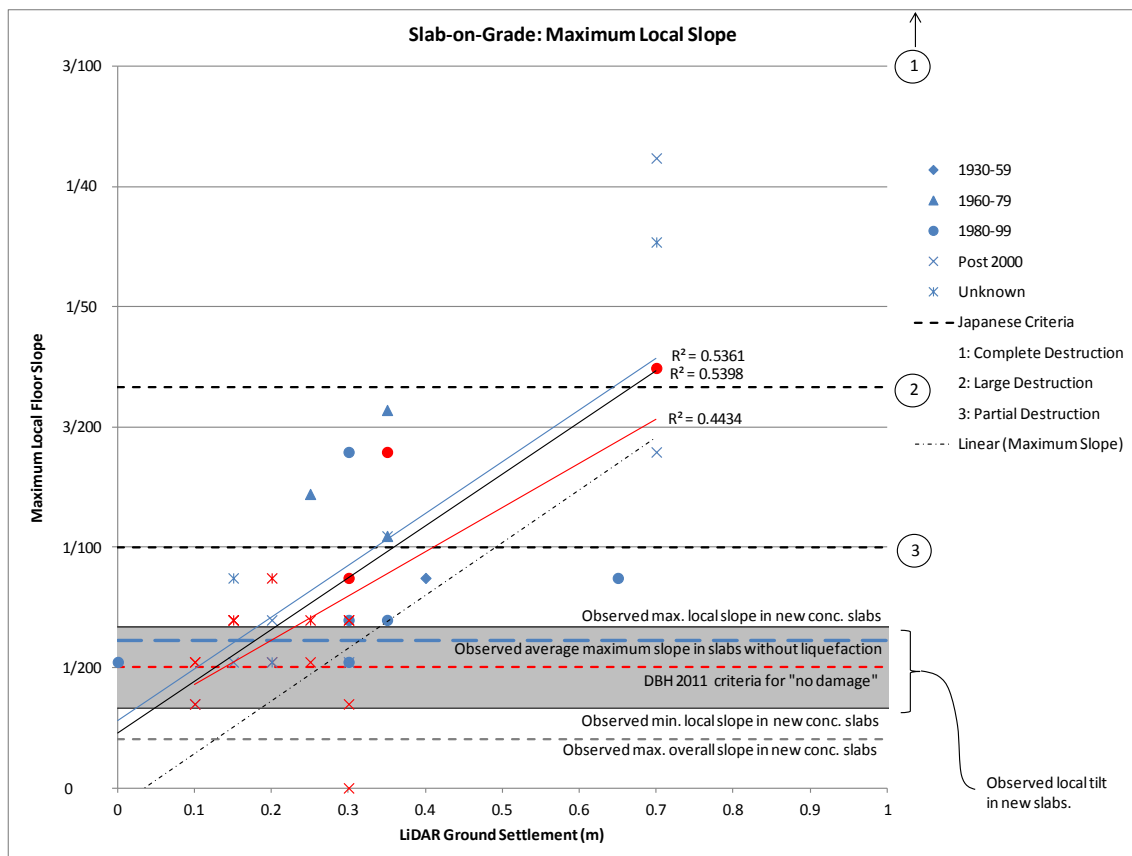


Figure A. 118. Maximum local floor slope vs. LiDAR ground settlement for slab-on-grade foundations, showing no damage references.

N.2.3: RibRaft

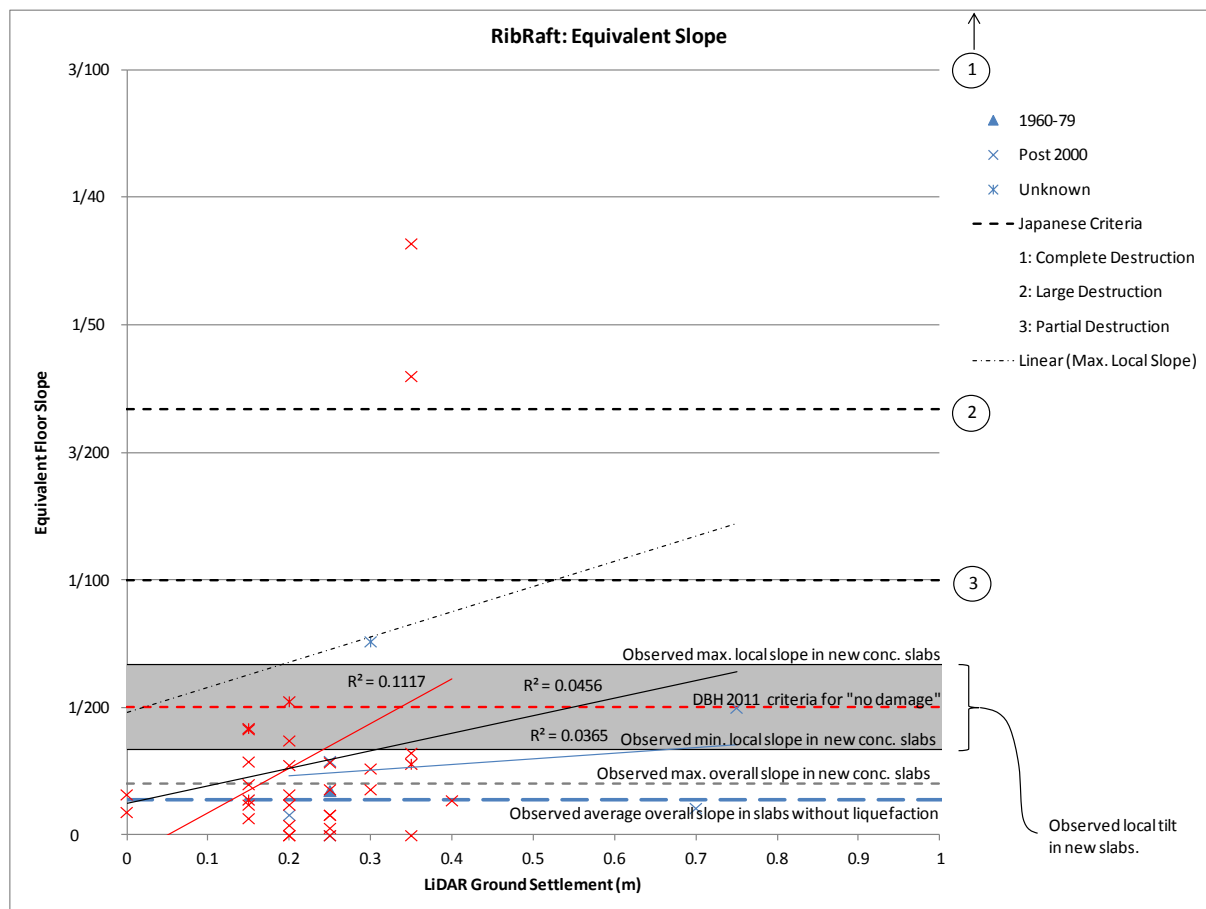


Figure A. 119. Equivalent floor slope vs. LiDAR ground settlement for ribraft foundations, showing no damage references.

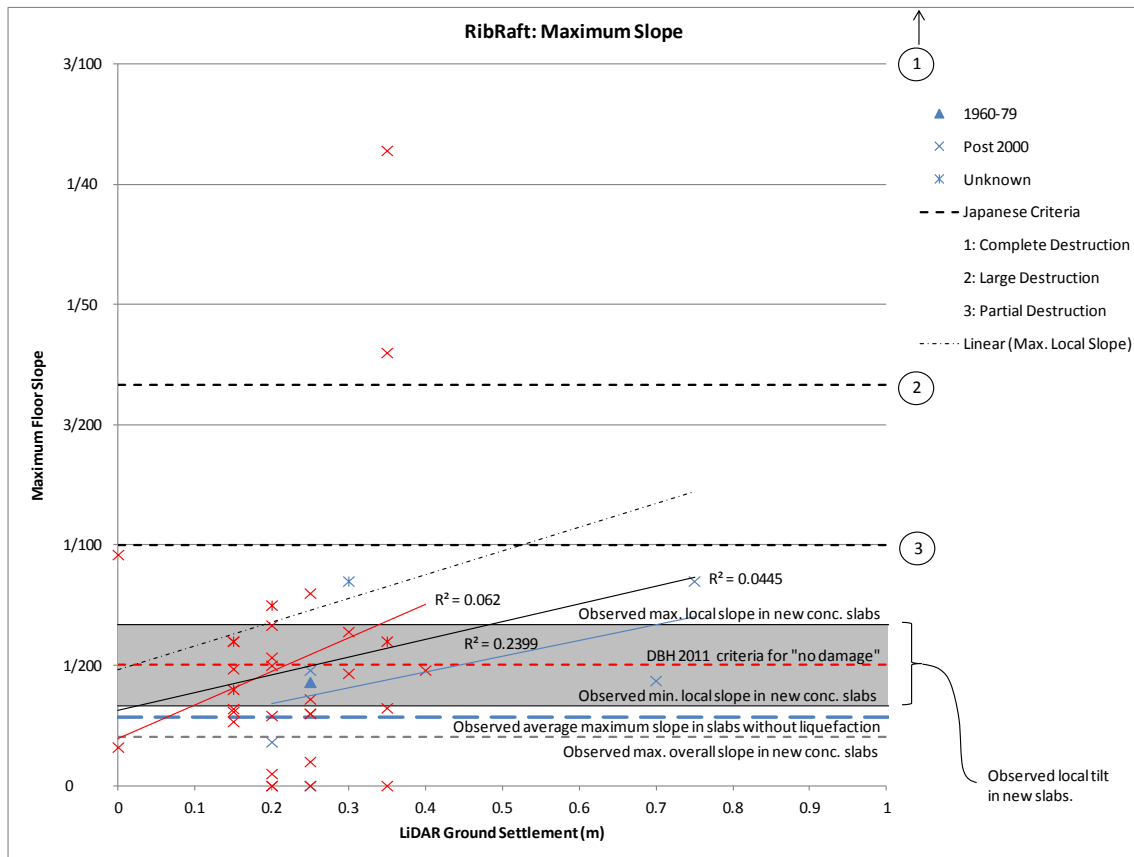


Figure A. 120. Maximum floor slope vs. LiDAR ground settlement for ribraft foundations, showing no damage references.

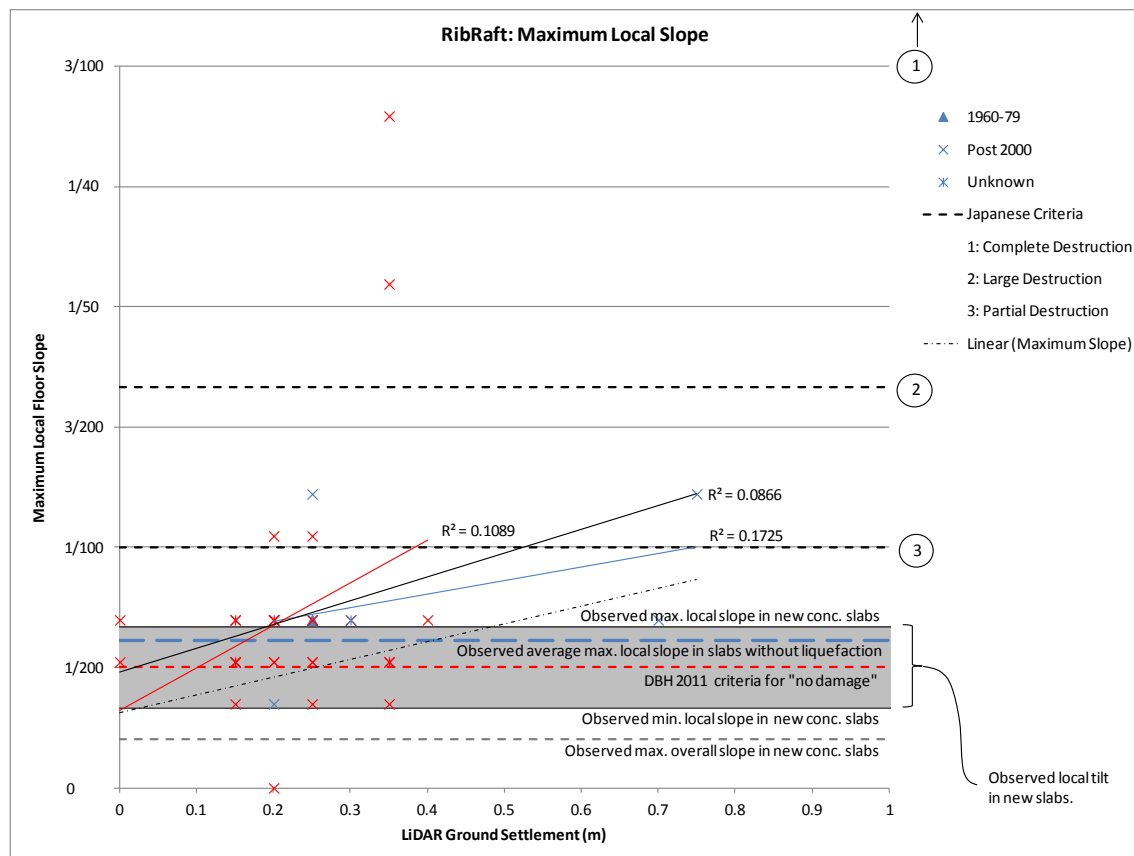


Figure A. 121. Maximum local floor slope vs. LiDAR ground settlement for ribraft foundations, showing no damage references.

N.2.4: Piled

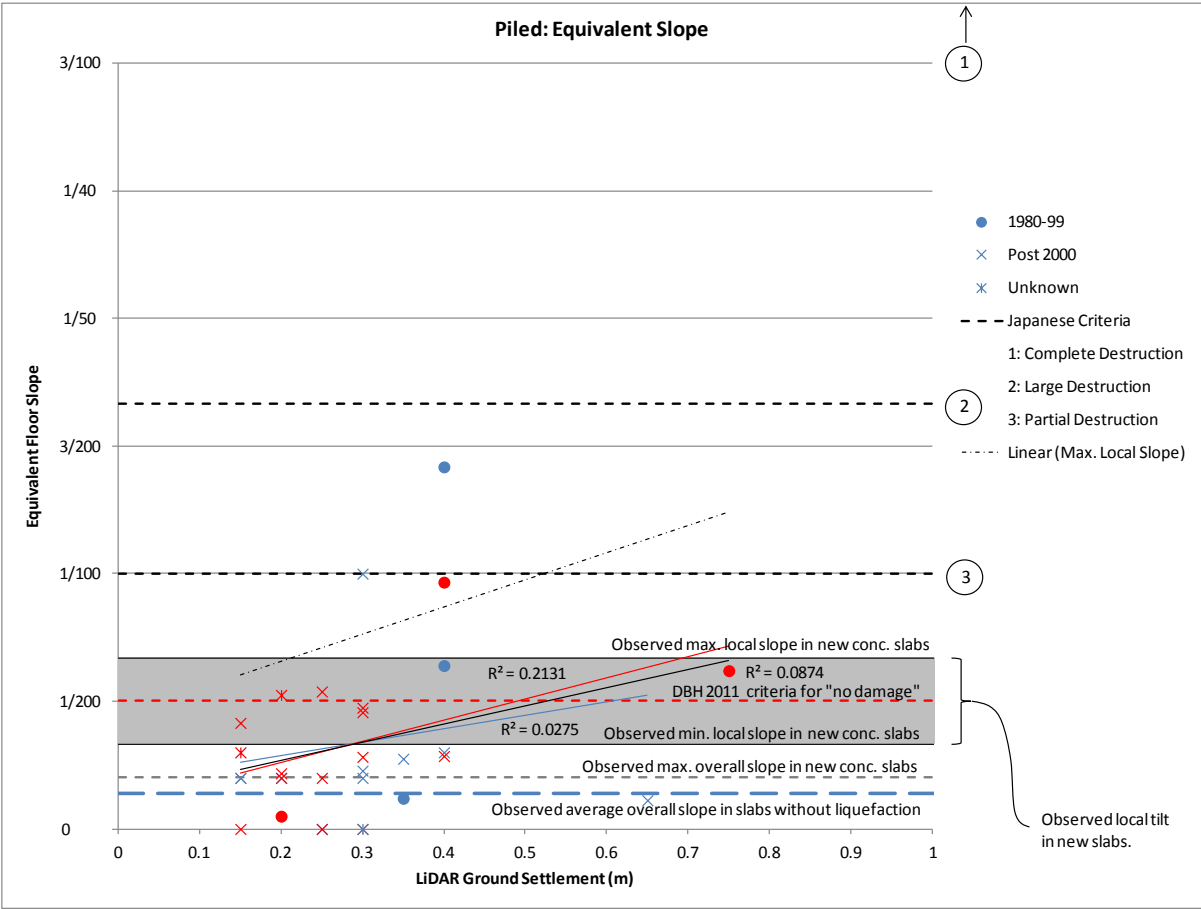


Figure A. 122. Equivalent floor slope vs. LiDAR ground settlement for piled foundations, showing no damage references.

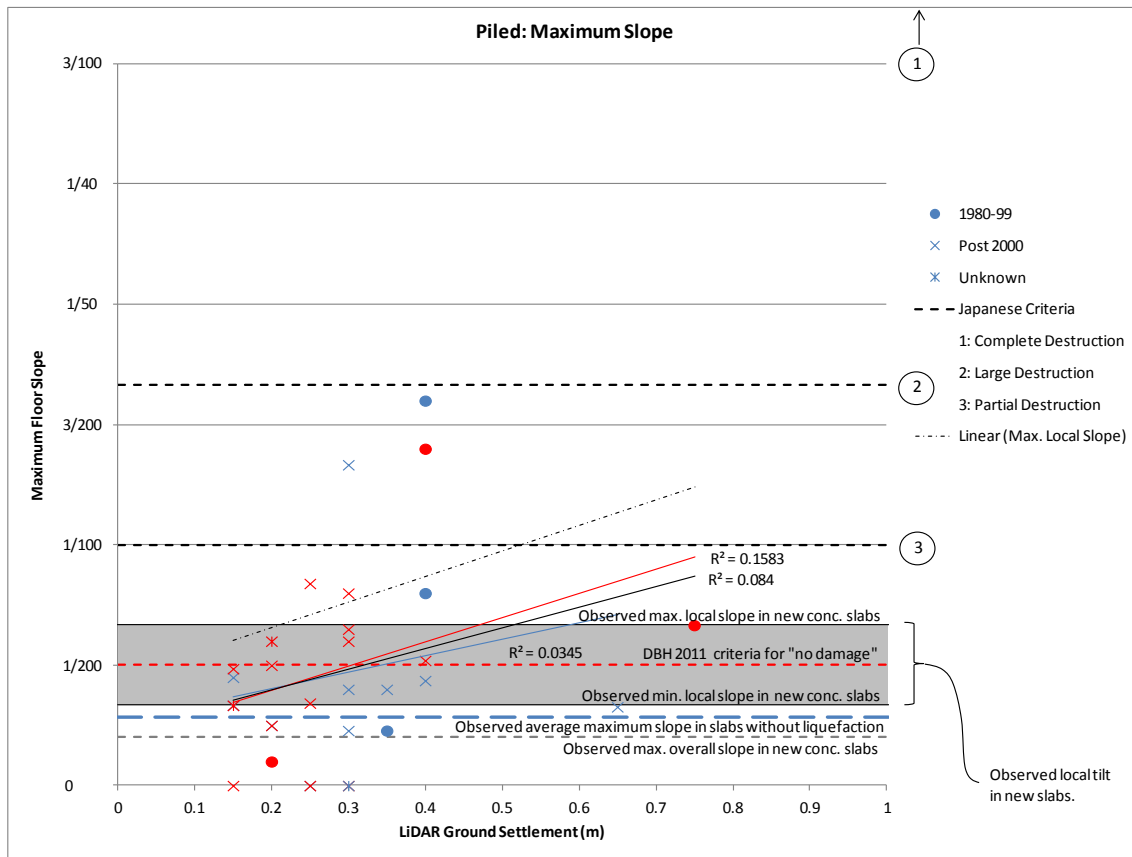


Figure A. 123. Maximum floor slope vs. LiDAR ground settlement for piled foundations, showing no damage references.

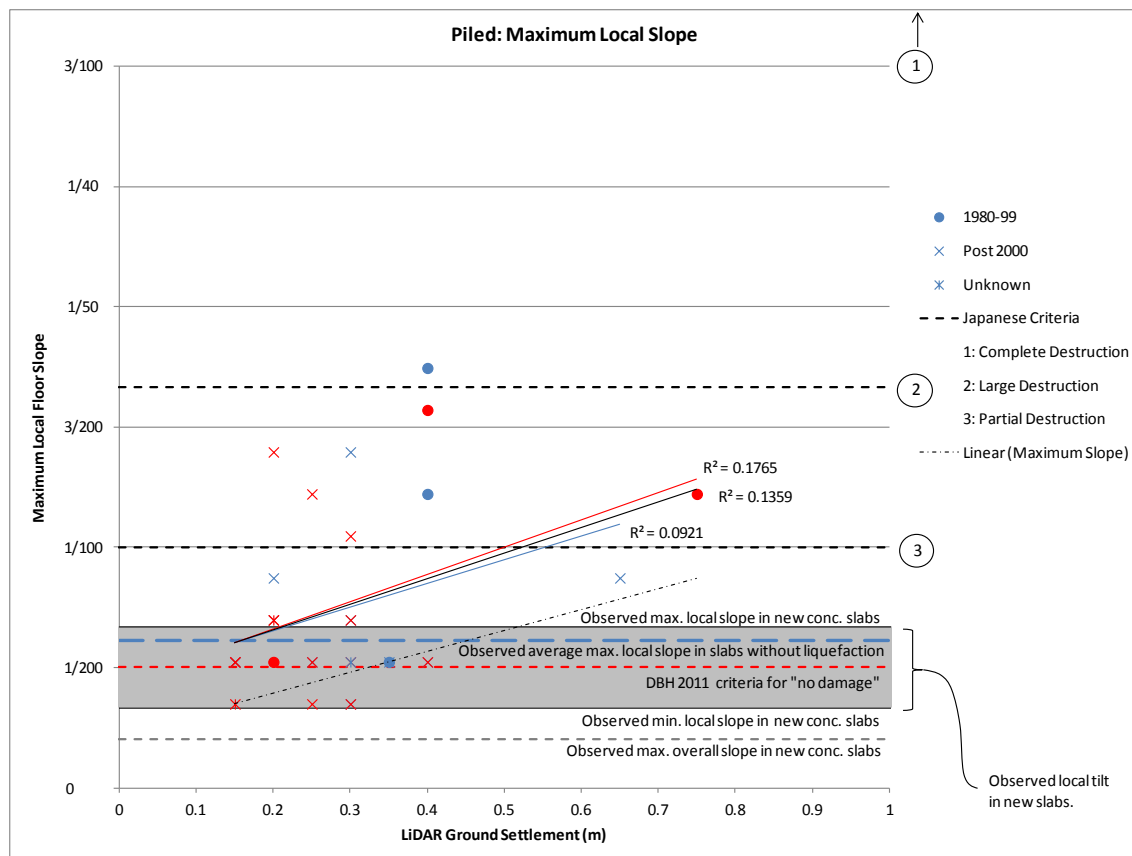


Figure A. 124. Maximum local floor slope vs. LiDAR ground settlement for piled foundations, showing no damage references.

N.2.5: Trendline Comparison

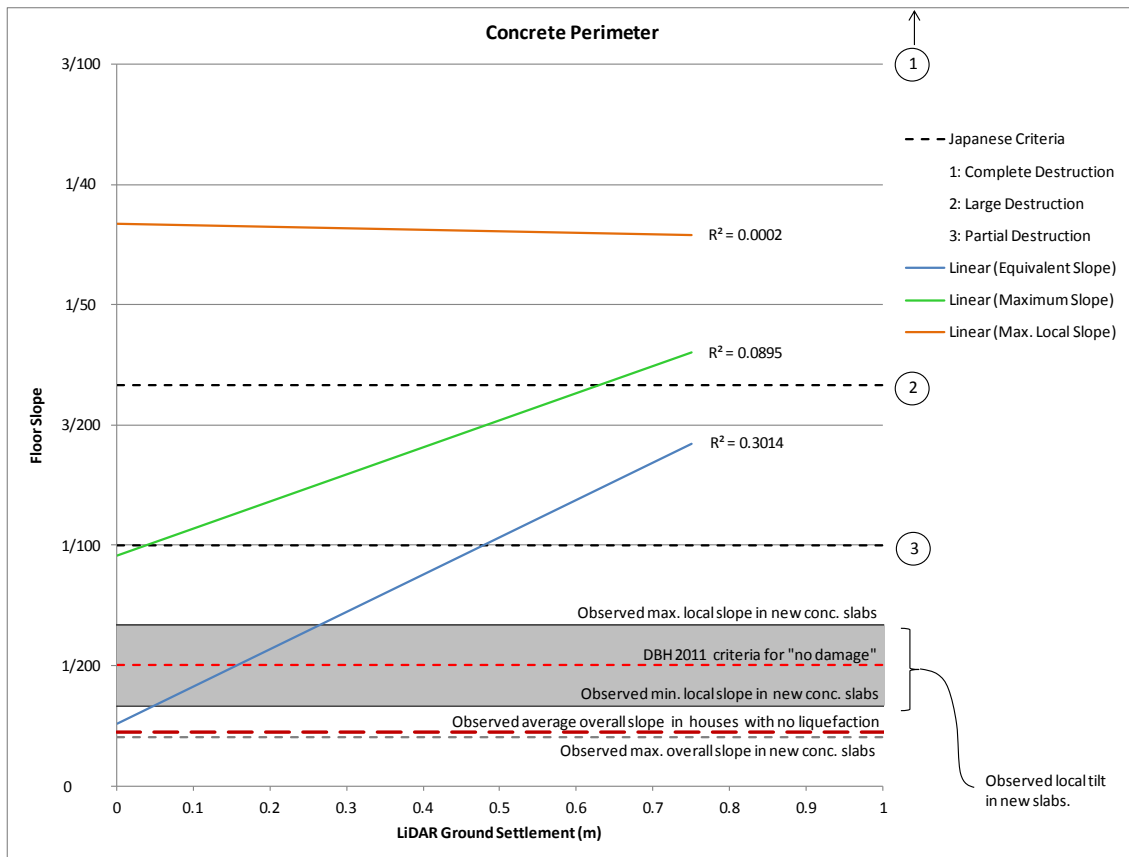


Figure A. 125. Trendlines for all floor slope indices vs. LiDAR ground settlement for concrete perimeter foundations.

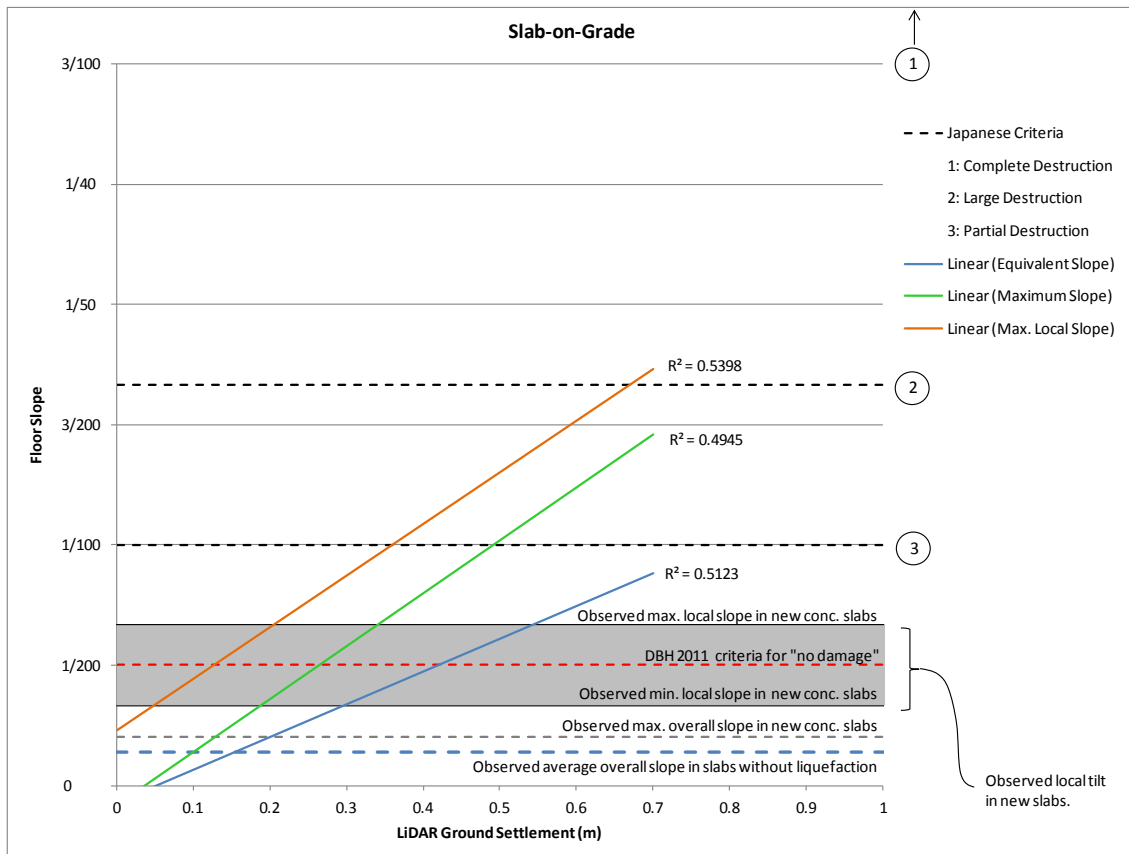


Figure A. 126. Trendlines for all floor slope indices vs. LiDAR ground settlement for slab-on-grade foundations.

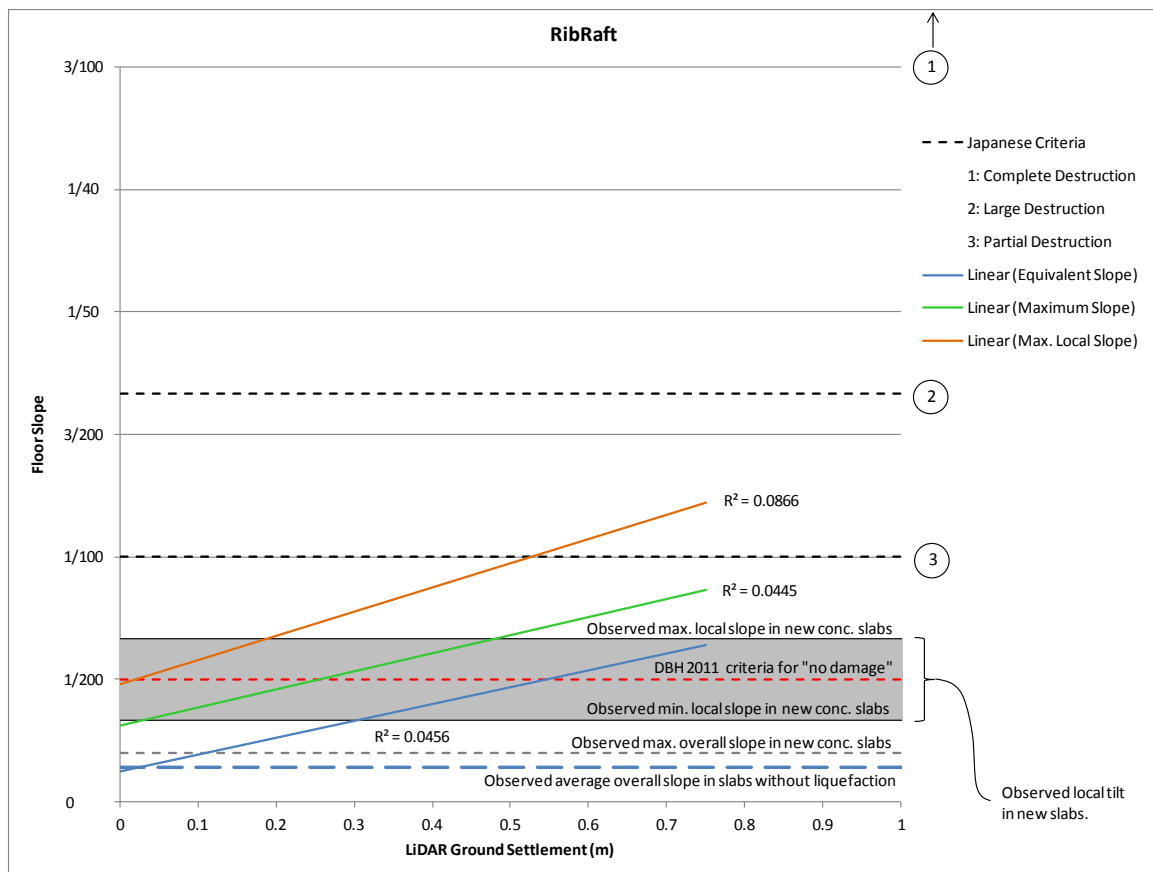


Figure A. 127. Trendlines for all floor slope indices vs. LiDAR ground settlement for ribraft foundations.

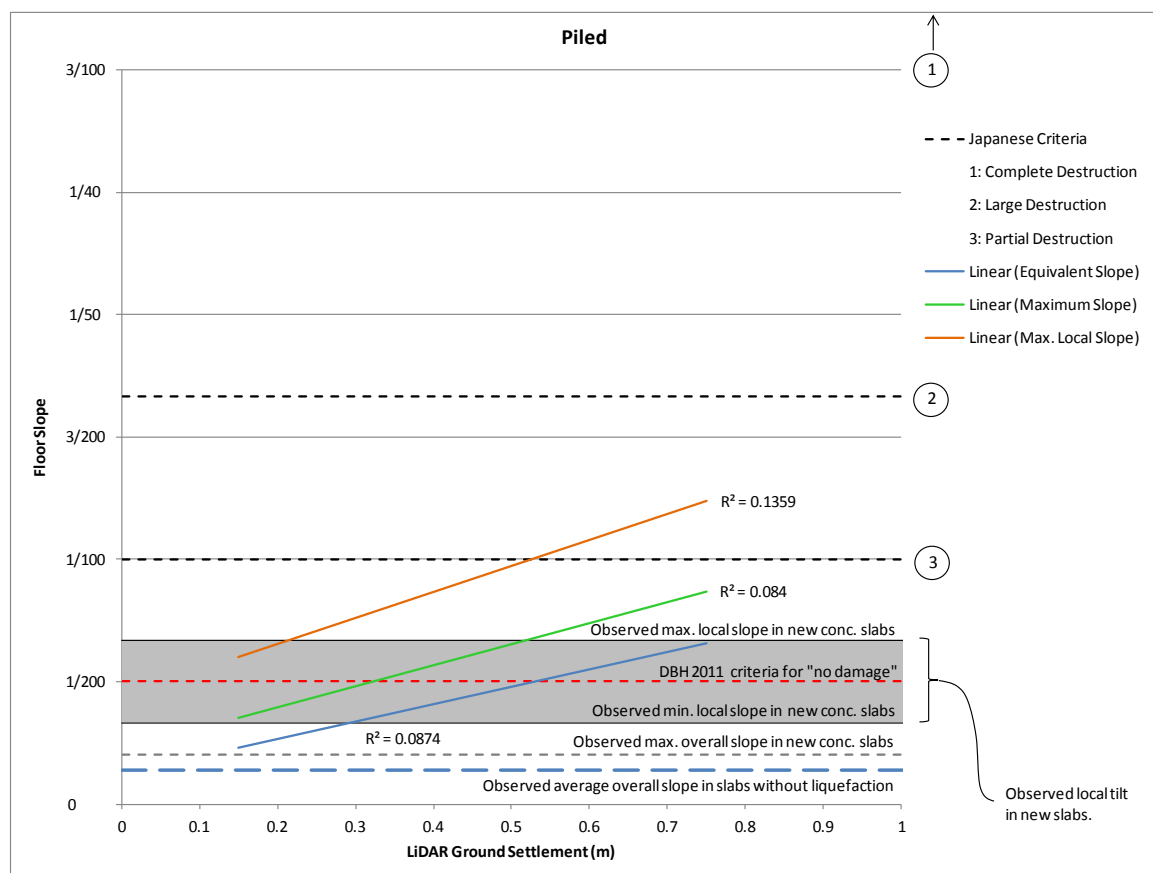


Figure A. 128. Trendlines for all floor slope indices vs. LiDAR ground settlement for piled foundations.

Appendix O: Section Modelling

Geometric Properties		
	Gross Conc.	Trans (n=9.72)
Area (mm ²) x 10 ³	90.0	90.0
Inertia (mm ⁴) x 10 ⁶	2700.0	2700.0
y _t (mm)	300	300
y _b (mm)	300	300
S _t (mm ³) x 10 ³	9000.0	9000.0
S _b (mm ³) x 10 ³	9000.0	9000.0

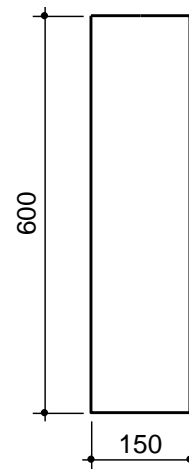
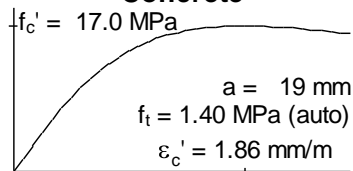
Crack Spacing

$$2 \times \text{dist} + 0.1 d_b / \rho$$

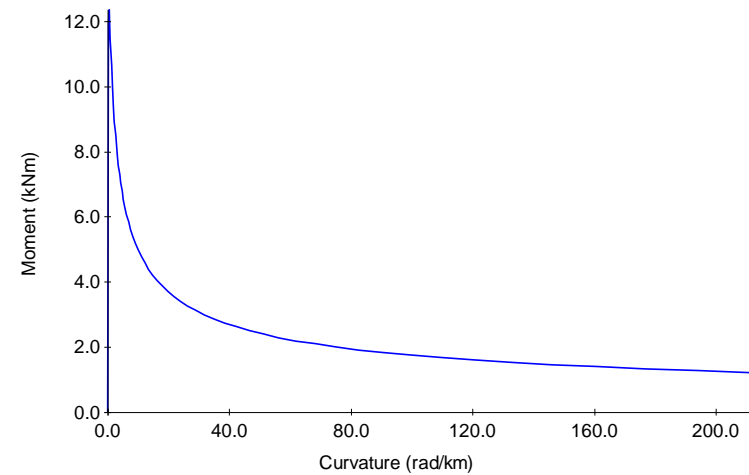
Loading (N.M.V + dN.dM.dV)

0.0 , -0.0 , 0.0 + 0.0 , 1.0 , 0.0

Concrete



Moment-Curvature

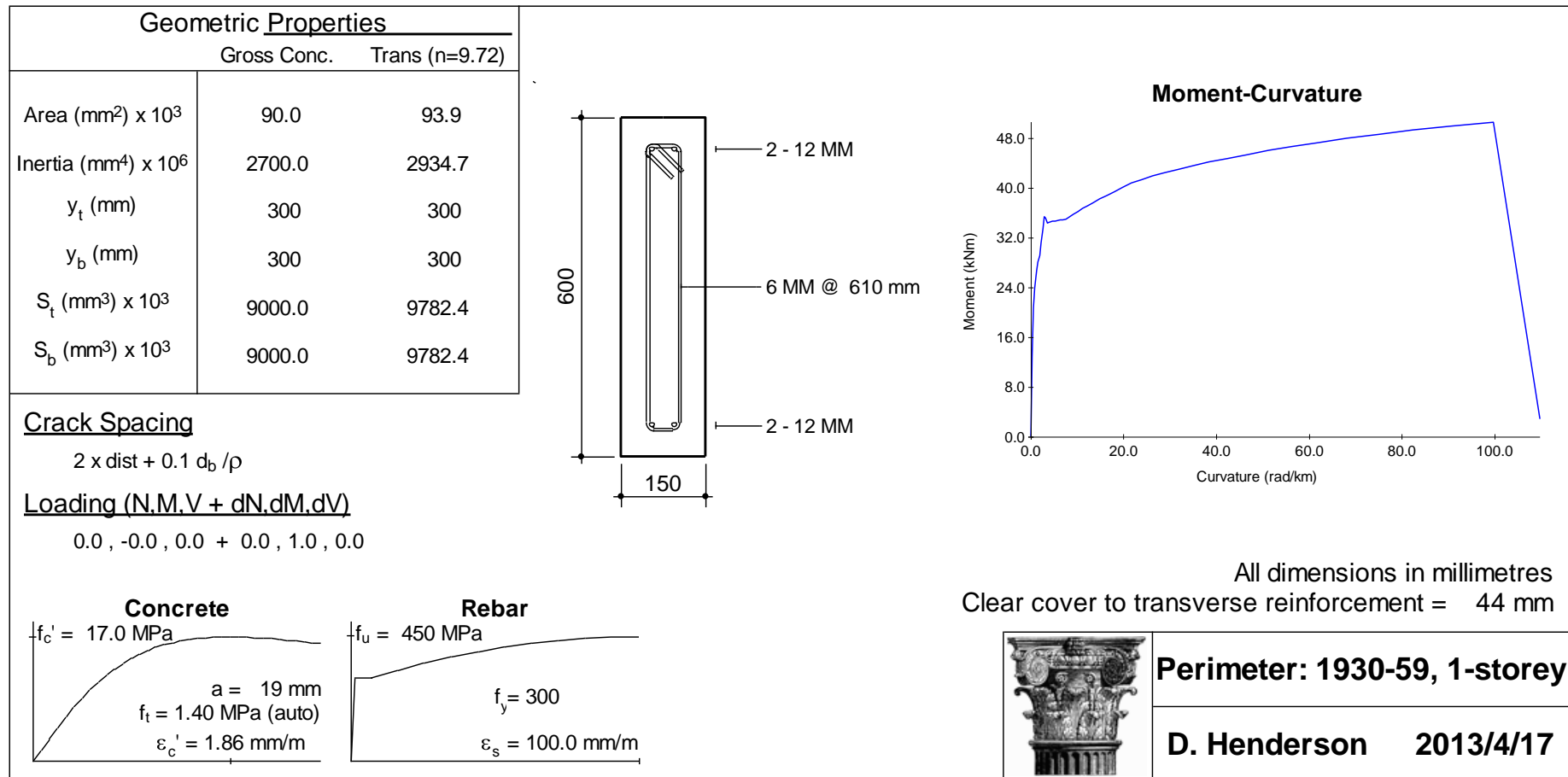


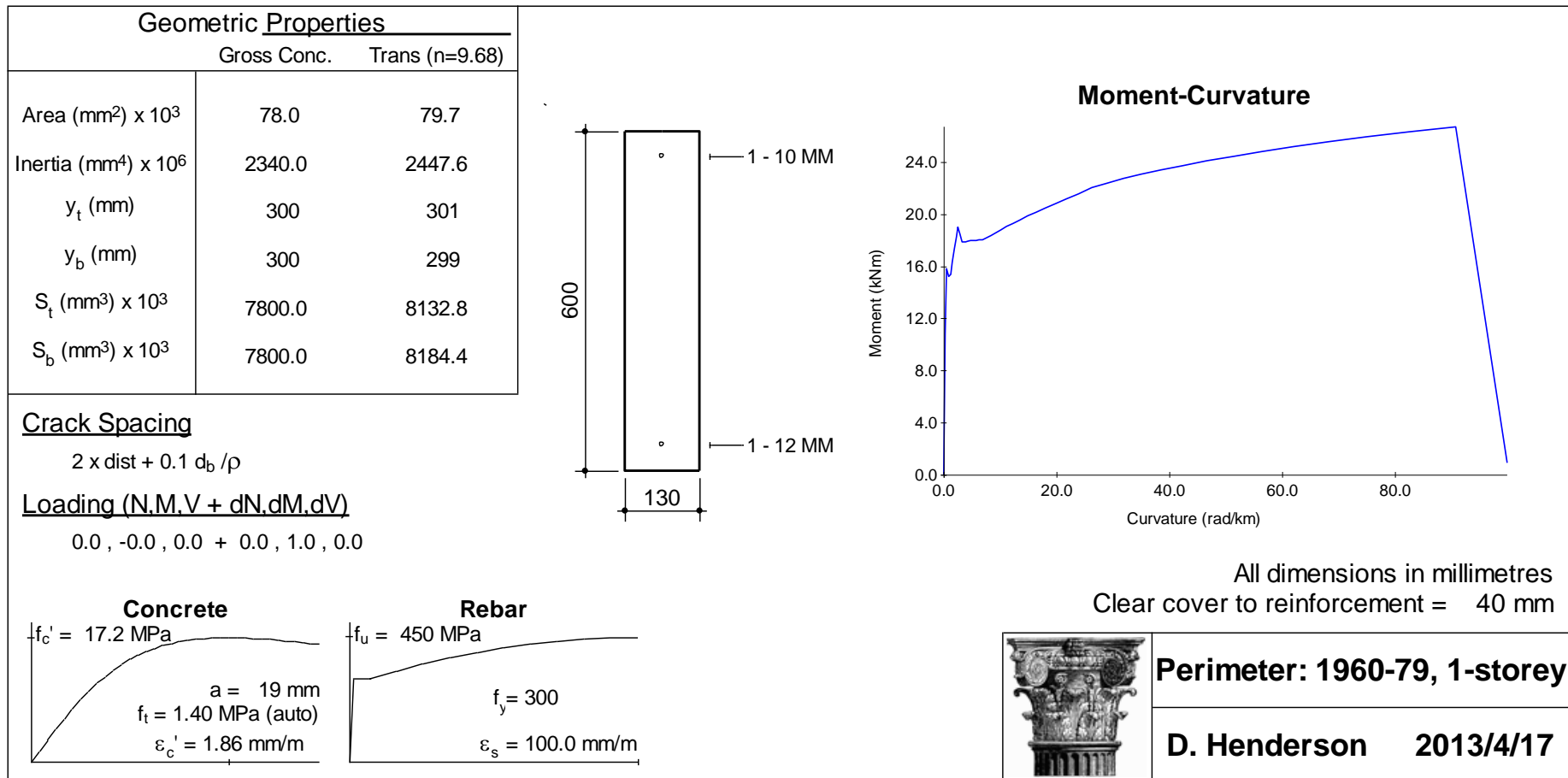
All dimensions in millimetres

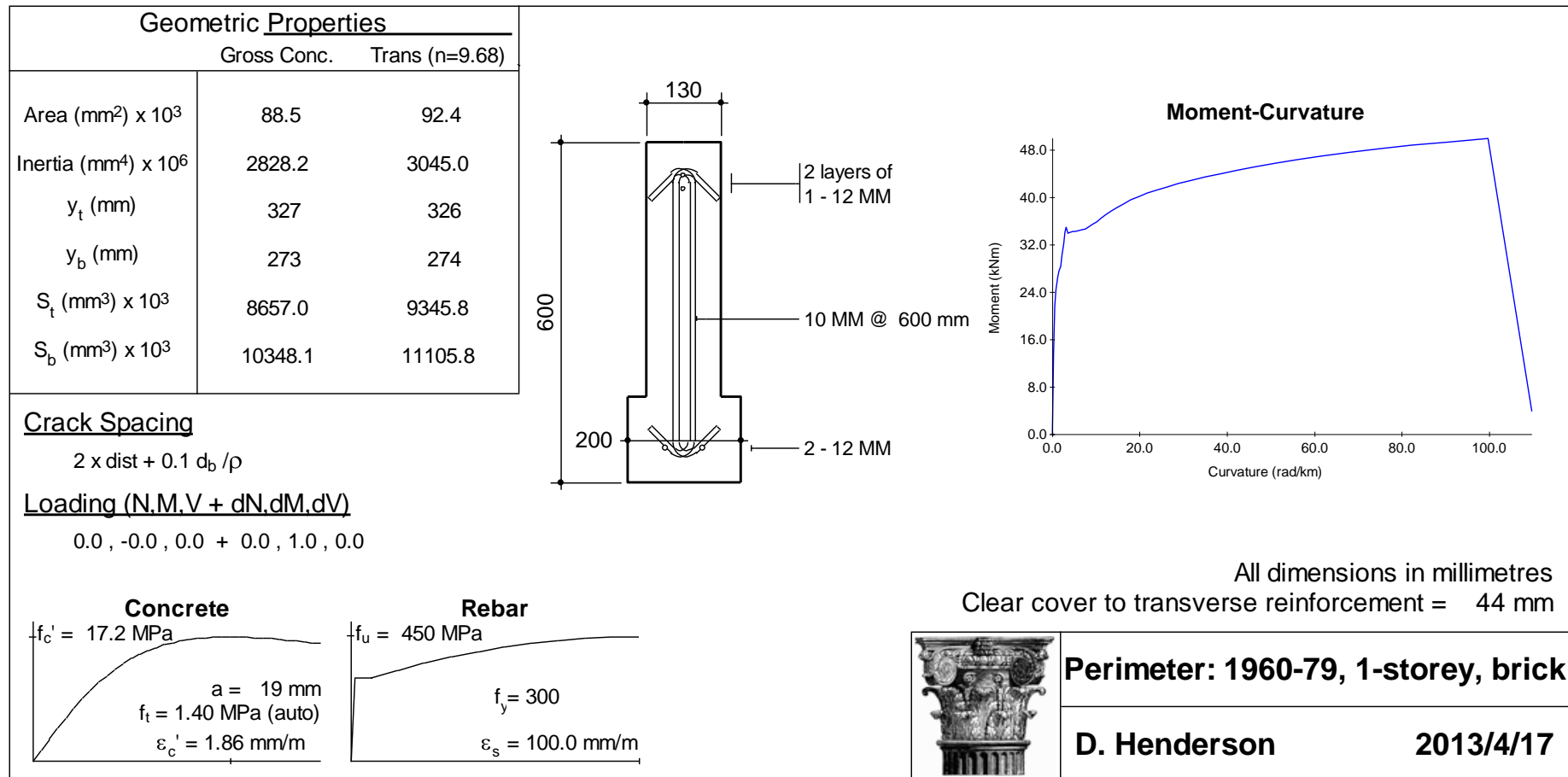


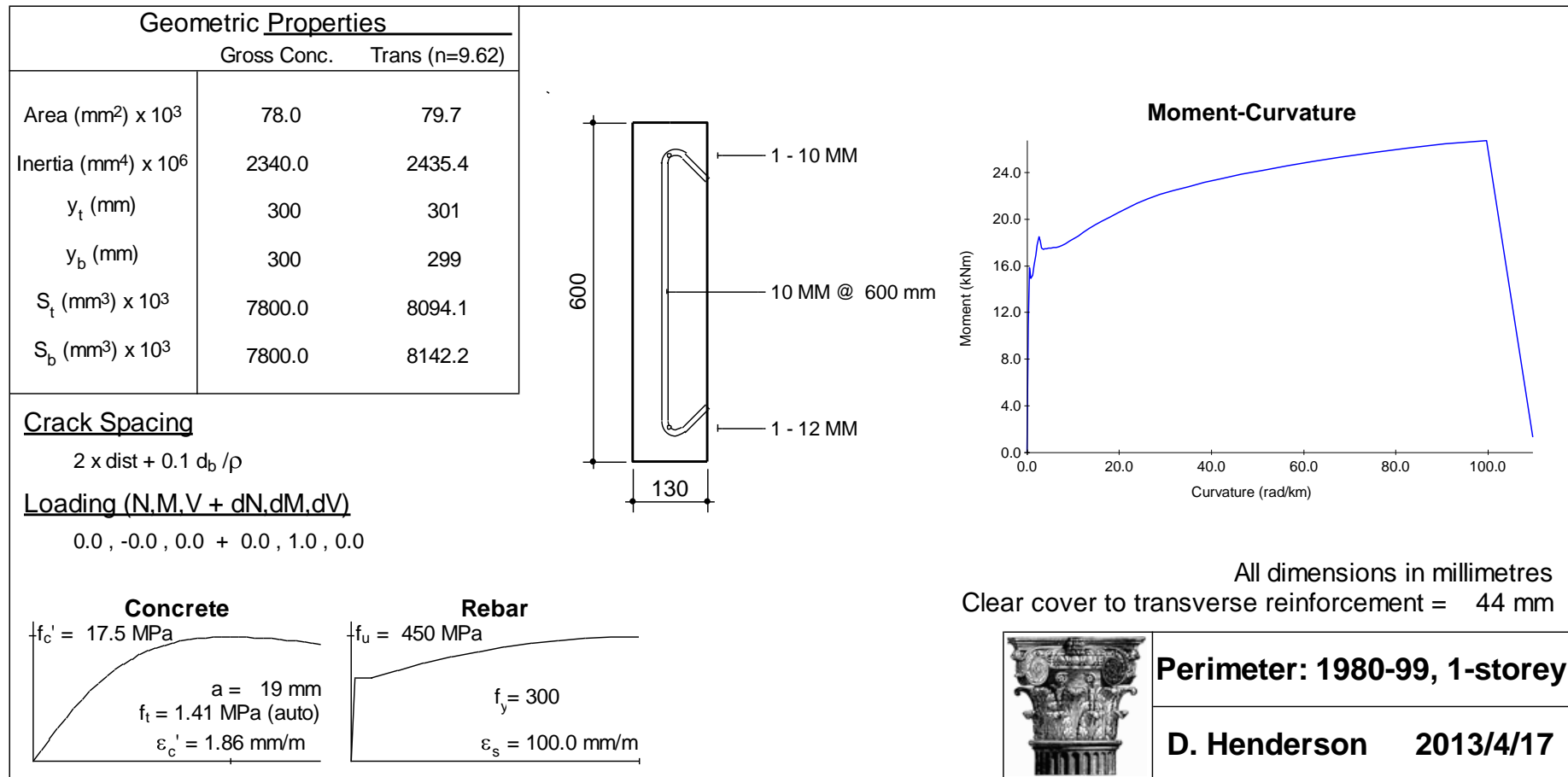
Perimeter: Pre-1930

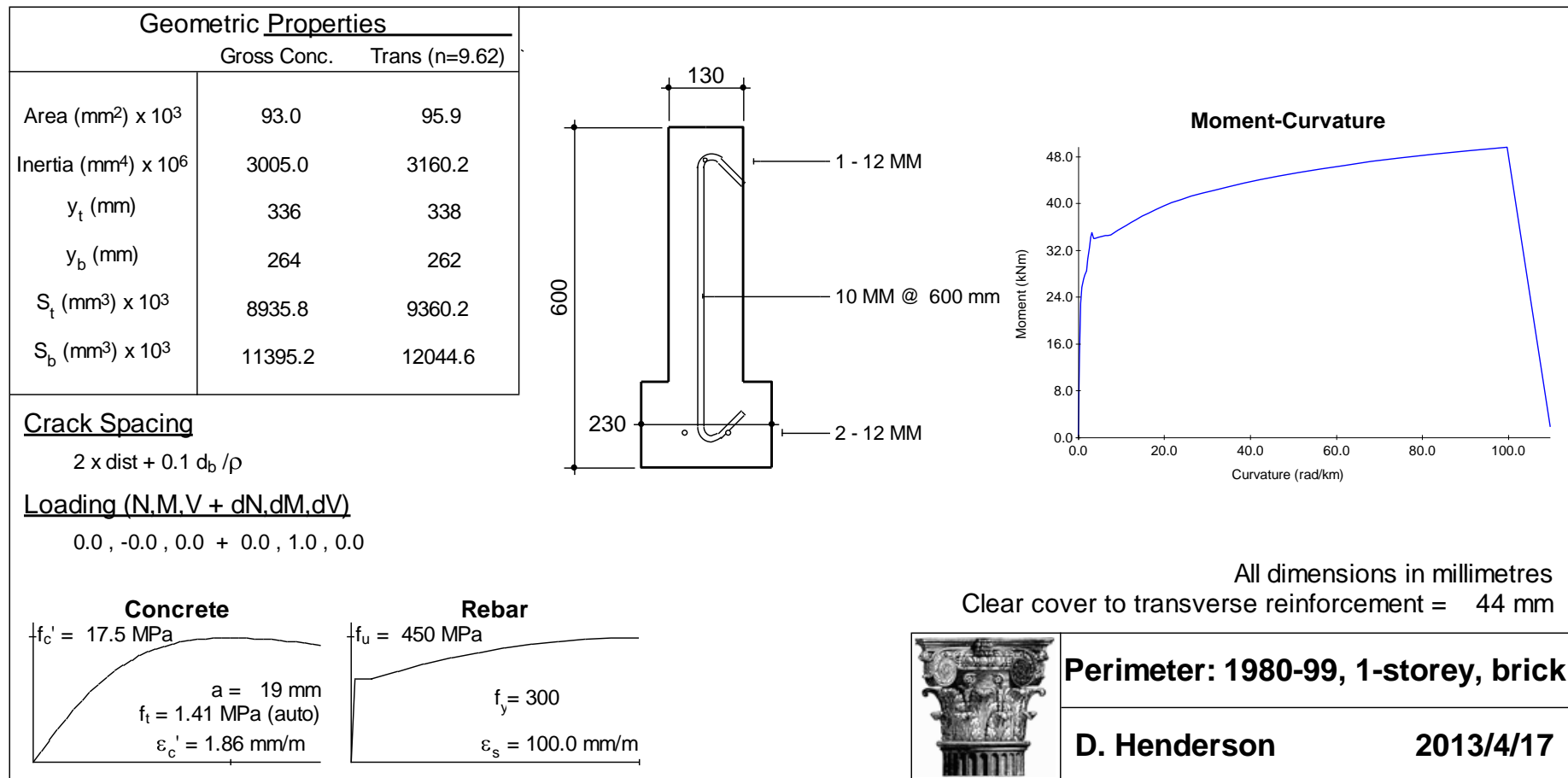
D. Henderson 2013/6/27

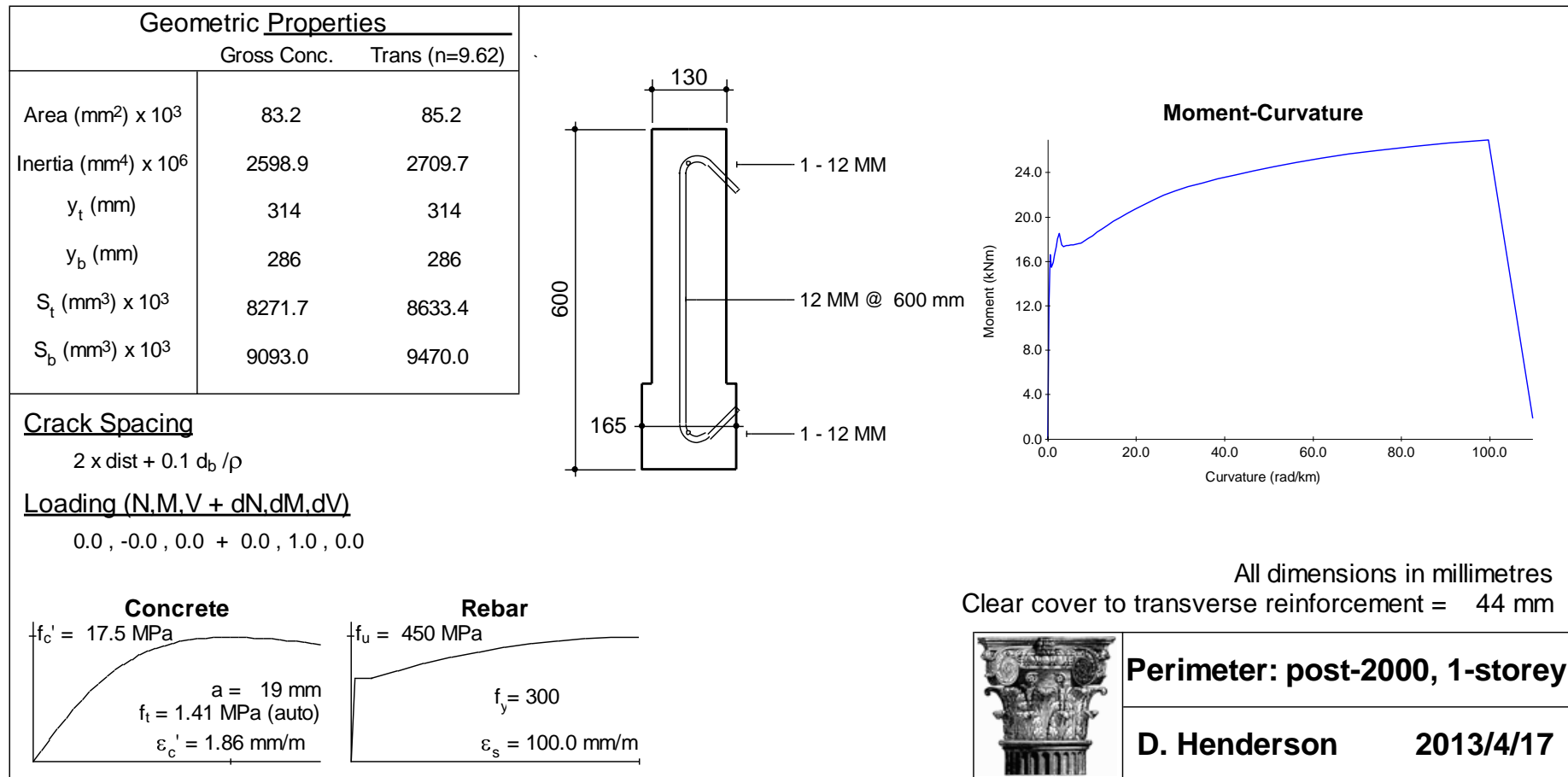


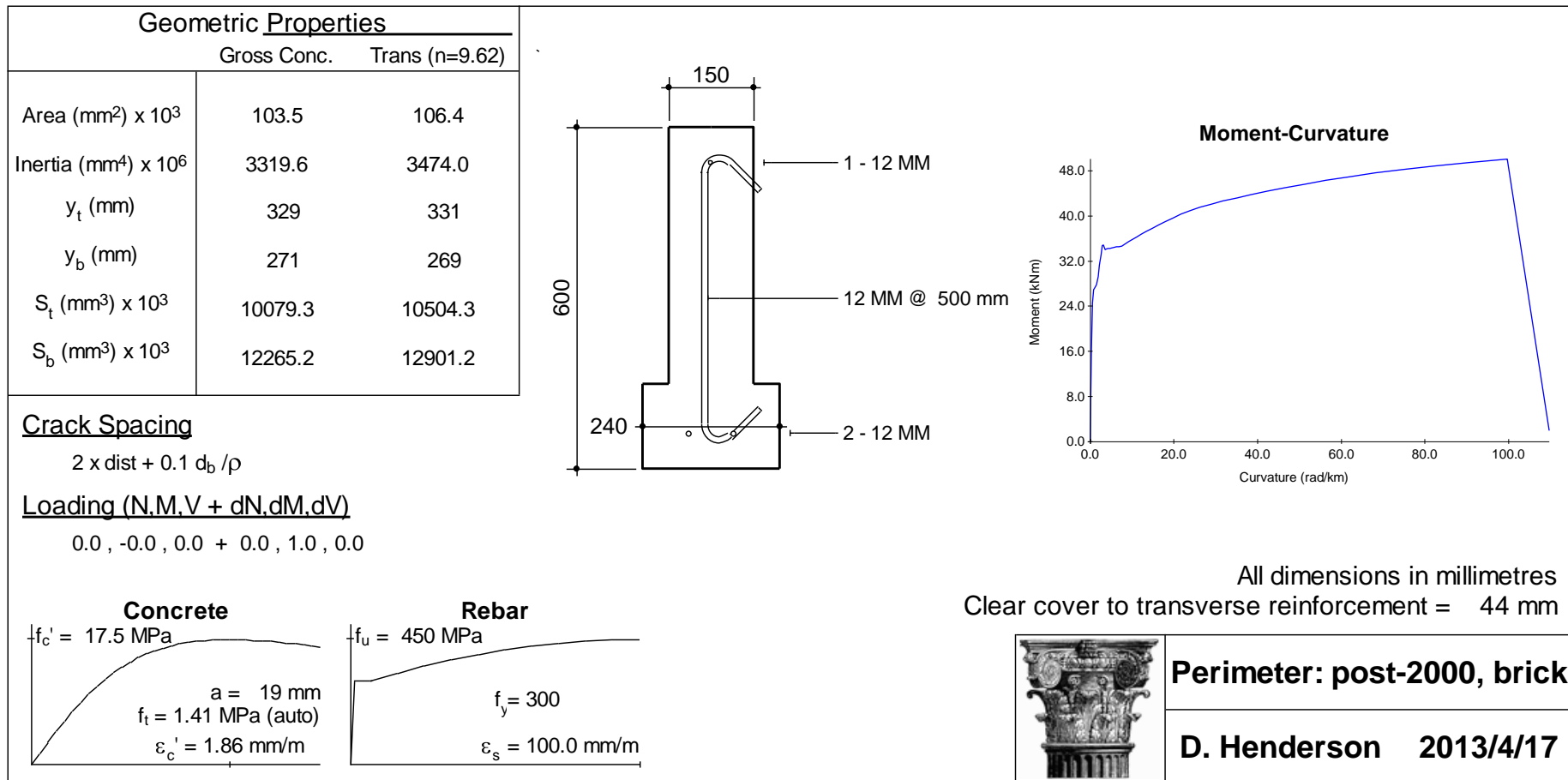












Appendix P: House Weight Calculations

Civil Engineering
Department

Project: FOUNDATION LOAD CALCS.

Subject: _____

By: Duncan Henderson

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FOUNDATION LOADS ON A CONCRETE PERIMETER

(example calculation)

HOUSE WEIGHTS

- Calculate the overall weight

of a 'standard' house →

- Floor area = $(14 \times 13) - (6 \times 6)$
 $= 146 \text{ m}^2$

Roof Weight

Heavy Roof

- 80 kg/m^2 (NZS 3604)

- say 25° pitch

→ roof area = $146 + (2 \times 13 + 2 \times 14) \times 0.5$
 $= 173 \text{ m}^2$

→ roof weight = $173 \times 1.1 \times 0.8$
 $= 152 \text{ kN}$

Light Roof

- 20 kg/m^2 (NZS 3604)

- roof area = 173 m^2

→ roof weight = $173 \times 1.1 \times 0.2 = 38 \text{ kN}$

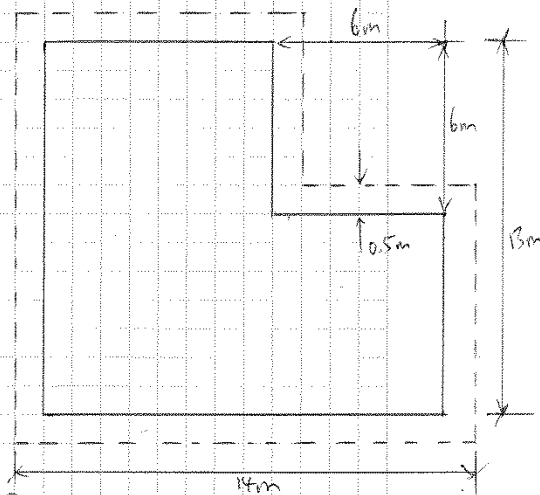
Roof Framing

- assume $100 \times 50 \text{ mm}$ timber, at 5.5 kg/m^3

- 0.15 m spacing equivalent for trusses + purlins

→ framing = $\frac{0.1 \times 0.05}{0.15} \times 5.5 = 0.18 \text{ kPa}$

weight = $0.18 \times 173 = 32 \text{ kN}$



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~ Ceiling

- framing, assume 100x50 mm timber, 0.45m spacing.

- ceiling jib board 0.12 kPa.

$$\rightarrow \text{framing weight} = \frac{0.1 \times 0.05}{0.45} \times 5.5 = 0.06 \text{ kPa}$$

$$\rightarrow \text{total weight} = (0.06 + 0.12) \times 146 = \underline{26 \text{ kN}}$$

~ Ground Floor

~ assume 125x50 mm timber, 0.45m spacing.

~ overlaid by 20mm particle board (0.16 kPa).

$$\rightarrow \text{framing weight} = \frac{0.125 \times 0.05}{0.45} \times 5.5 = 0.08 \text{ kPa}$$

$$\rightarrow \text{total weight} = (0.08 + 0.16) \times 146 = \underline{34.5 \text{ kN}}$$

~ First Floor

~ assume 200x50 mm timber, 0.6m spacing

~ overlaid by 20mm particle board (0.16 kPa)
 10mm plaster board (0.12 kPa)

$$\rightarrow \text{framing weight} = \frac{0.2 \times 0.05}{0.6} \times 5.5 = 0.1 \text{ kPa}$$

$$\rightarrow \text{total weight} = (0.1 + 0.16 + 0.12) \times 146 = \underline{55.5 \text{ kN}}$$

- External Walls

~ Heavy Cladding (for a single storey).

- 220 kg/m² (NZS 3604)

$$\text{weight} = 220 \times 2.4 \times 54 \text{ m} = 285 \text{ kN}$$

$$\text{framing} = 0.06 \times 2.4 \times 54 \text{ m} = 8 \text{ kN}$$

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~ Light Cladding

- 20 kg/m^2 (NZS 3604)

~ weight = $20 \times 2.4 \times 54 = 26 \text{ kN}$

~ framing = 8 kN

~ Partitions

~ 2x plaster board, 0.12 kPa

~ 1x framing, 0.06 kPa

~ guess 50m of walls,

~ weight = $(2 \times 0.12 + 0.06) \times 2.4 \times 50 = \underline{36 \text{ kN}}$

Adding up total house weights:

	Light Light	Light Heavy	Heavy Light	Heavy Heavy
Roof: Cladding:				
Roof:	38	38	152	152
Roof framing:	32	32	32	32
Ceiling:	26	26	26	26
Ground floor	35	35	35	35
External walls:	26	285	26	285
	8	8	8	8
Partitions:	36	36	36	36
Total weight:	201 kN	460 kN	315 kN	574 kN
÷ 54m				
= Perimeter load:	3.7 kN/m	8.5 kN/m	5.8 kN/m	10.6 kN/m

Appendix Q: Soil Stiffness Calculations

**Civil Engineering
Department**

Project: SOIL PARAMETER CALCS.
Subject: _____
By: DUNCAN HENDERSON
Date: _____ Page 1 of 1



SOIL STIFFNESS PARAMETER FOR MODELLING.

SUBGRADE REACTION COEFFICIENT.

Equations:

$$k_{B \times B} = k_{0.3} \left(\frac{B + 0.3}{2B} \right)^2$$

$$k = \frac{k_{B \times B} (1 + 0.5 \frac{B}{L})}{1.5}$$

$$\rightarrow k = \frac{k_{0.3} \left(\frac{B + 0.3}{2B} \right)^2 (1 + 0.5 \frac{B}{L})}{1.5}$$

with: $B = \text{width of foundation} = 0.2 \text{ m}$

$L = \text{length of foundation} = 10 \text{ m}$

$k_{0.3} = 25000 \text{ kN/m}^3$, chosen from Table 6.2, Das (2004)

$$\rightarrow k = \frac{25000 \left(\frac{0.2 + 0.3}{2 \times 0.2} \right)^2 (1 + 0.5 \frac{0.2}{10})}{1.5}$$

$$= \frac{25000 \times 1.5625 \times 1.01}{1.5}$$

$$= 26302.08 \text{ kN/m}^3$$

converting the footing width:

$$k_{\text{footing}} = 26302.08 \times 0.2$$

$$= 5260.4 \text{ kN/m}^2$$

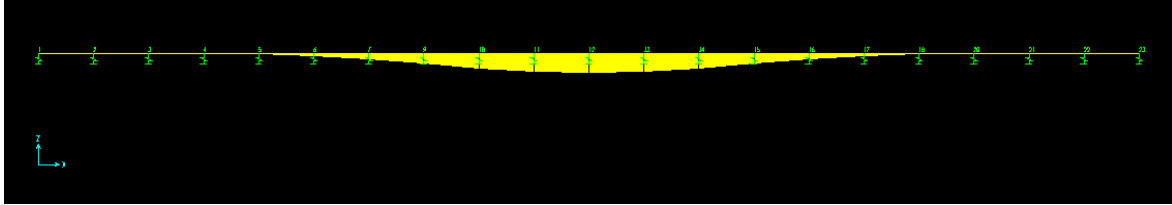
$$\rightarrow \text{use } k_{\text{footing}} = 5000 \text{ kN/m}^2$$

due to inherent approximations

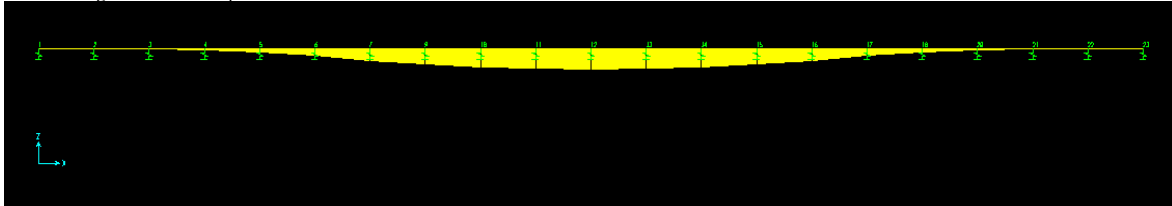
Appendix R: Bending Moment Diagram Examples

R.1: Bearing Loss in Centre, 80% Loss

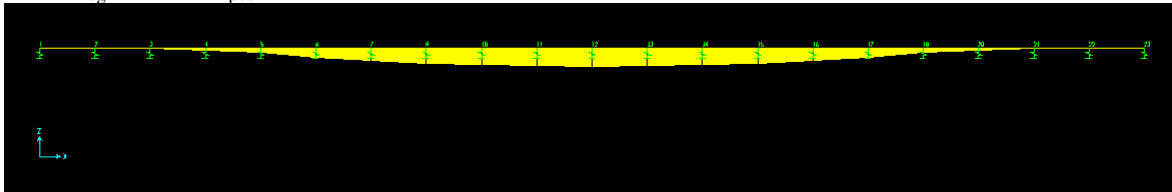
2m: $M_{\text{neg}} = \text{N/A}$, $M_{\text{pos}} = 17.8066\text{kNm}$



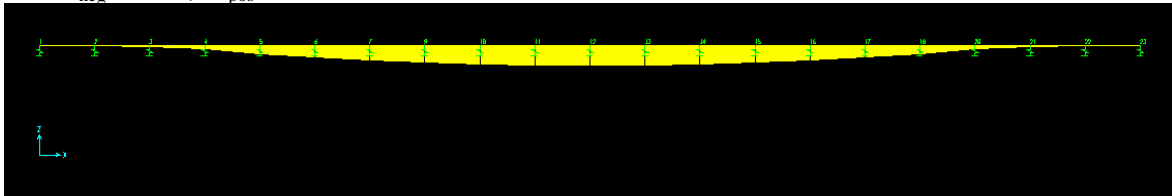
4m: $M_{\text{neg}} = \text{N/A}$, $M_{\text{pos}} = 30.245\text{kNm}$



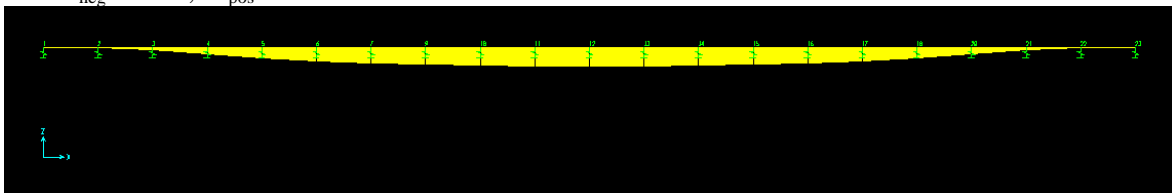
5m: $M_{\text{neg}} = \text{N/A}$, $M_{\text{pos}} = 34.4877\text{kNm}$



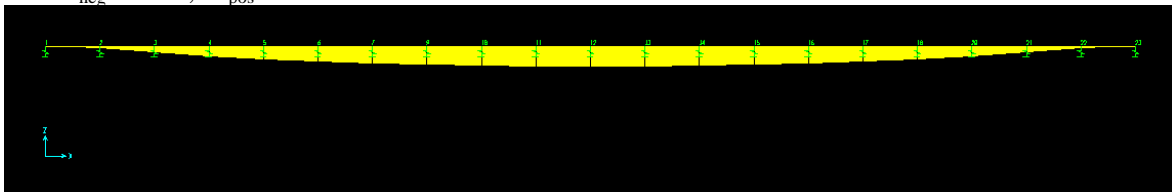
6m: $M_{\text{neg}} = \text{N/A}$, $M_{\text{pos}} = 36.4819\text{kNm}$



7m: $M_{\text{neg}} = \text{N/A}$, $M_{\text{pos}} = 35.4794\text{kNm}$

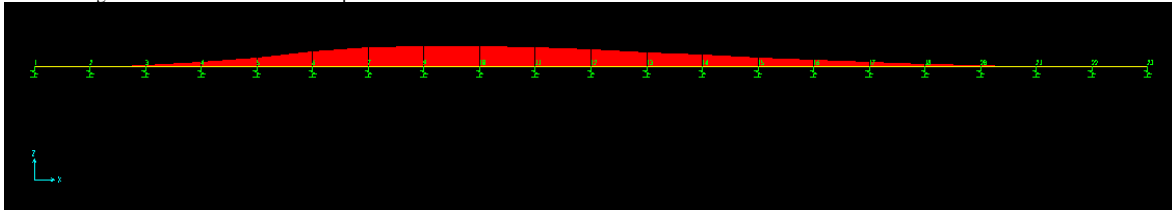


8m: $M_{\text{neg}} = \text{N/A}$, $M_{\text{pos}} = 30.2248\text{kNm}$

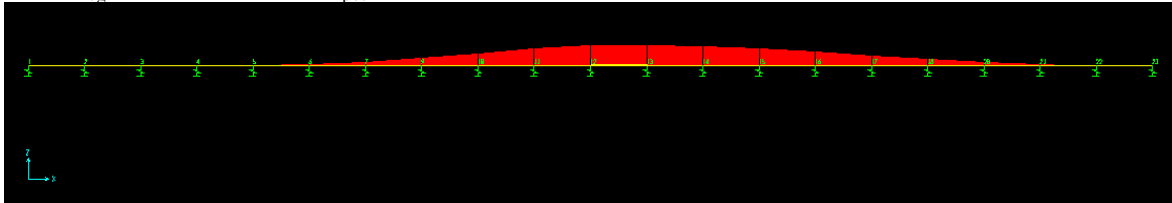


R.2: Bearing Loss at One End, 80% loss

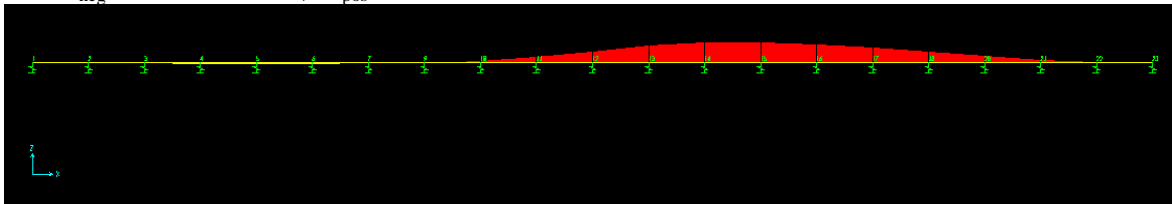
2m: $M_{\text{neg}} = -21.1721\text{kNm}$, $M_{\text{pos}} = \text{N/A}$



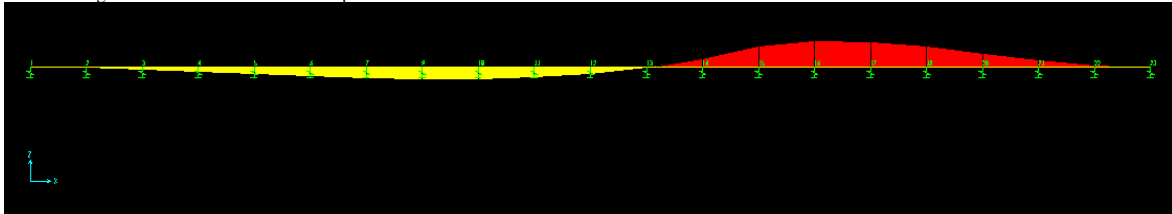
4m: $M_{\text{neg}} = -24.3127\text{kNm}$, $M_{\text{pos}} = 0.0349\text{kNm}$



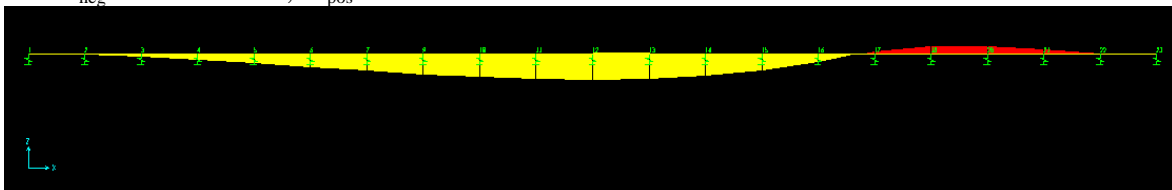
5m: $M_{\text{neg}} = -19.1902\text{kNm}$, $M_{\text{pos}} = 1.3713\text{kNm}$



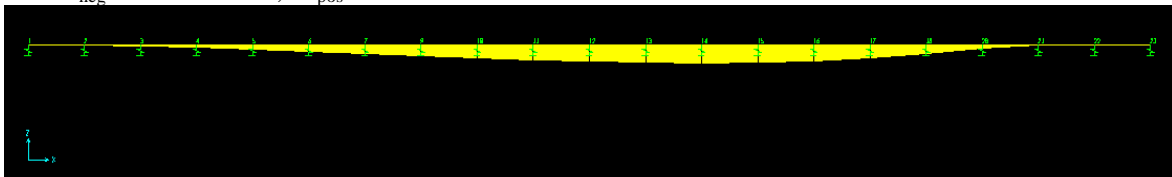
6m: $M_{\text{neg}} = -11.7148\text{kNm}$, $M_{\text{pos}} = 5.5211\text{kNm}$



7m: $M_{\text{neg}} = -3.8598\text{kNm}$, $M_{\text{pos}} = 11.4391\text{kNm}$

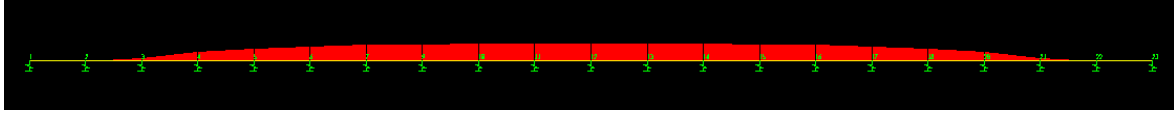


8m: $M_{\text{neg}} = 0.1631\text{kNm}$, $M_{\text{pos}} = 16.2549\text{kNm}$

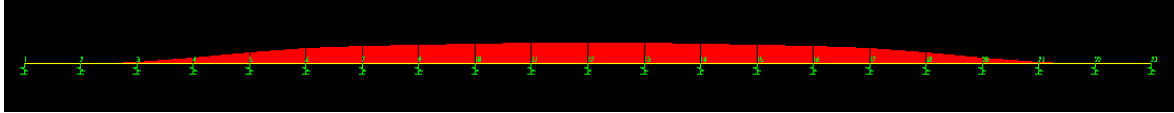


R.3: Bearing Loss at Both Ends, 80% loss

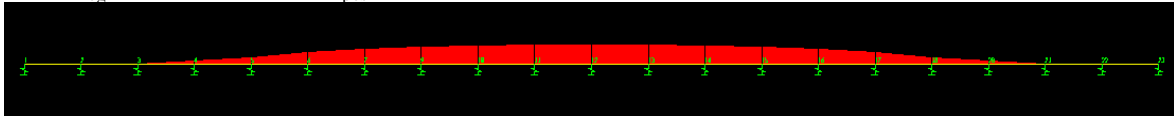
2m: $M_{neg} = -16.1606\text{kNm}$, $M_{pos} = \text{N/A}$



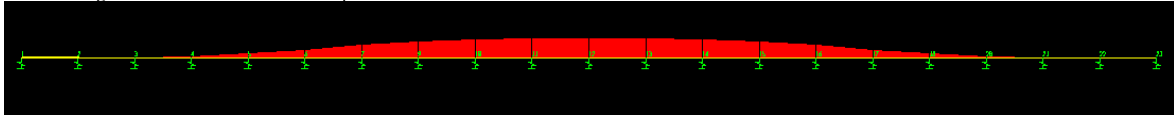
3m: $M_{neg} = -23.8075\text{kNm}$, $M_{pos} = \text{N/A}$



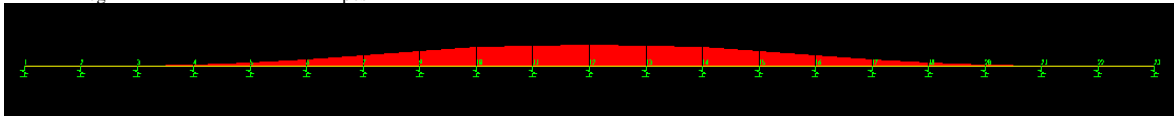
4m: $M_{neg} = -30.8405\text{kNm}$, $M_{pos} = \text{N/A}$



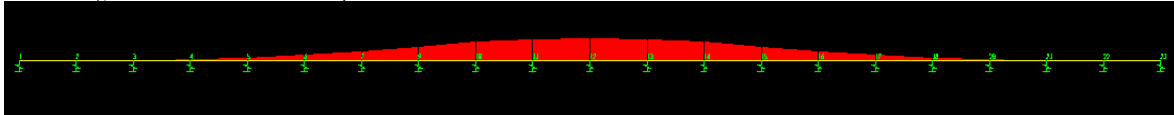
5m: $M_{neg} = -36.4622\text{kNm}$, $M_{pos} = \text{N/A}$



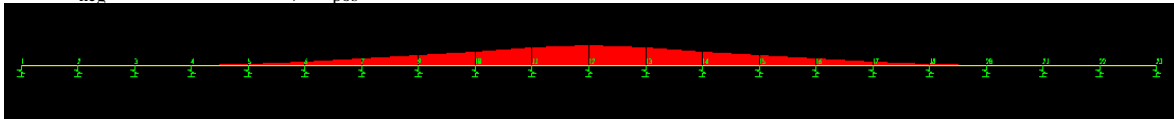
6m: $M_{neg} = -40.0023\text{kNm}$, $M_{pos} = \text{N/A}$



7m: $M_{neg} = -40.5526\text{kNm}$, $M_{pos} = \text{N/A}$



8m: $M_{neg} = 35.9989\text{kNm}$, $M_{pos} = \text{N/A}$



Appendix S: Beam Modelling Results

S.1: Bearing Loss in Centre

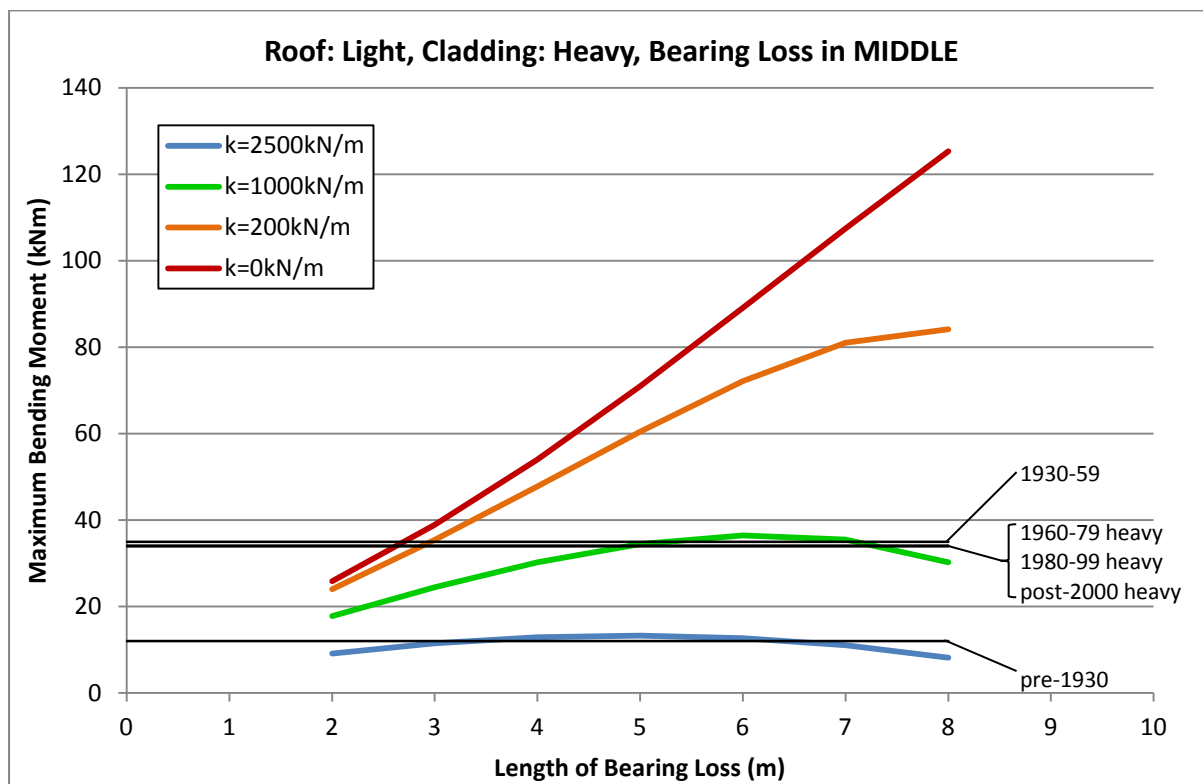


Figure A. 129. Moment demands for light roof and heavy cladding for different levels and lengths of bearing loss in centre. Yield capacities of the corresponding perimeter sections are also shown.

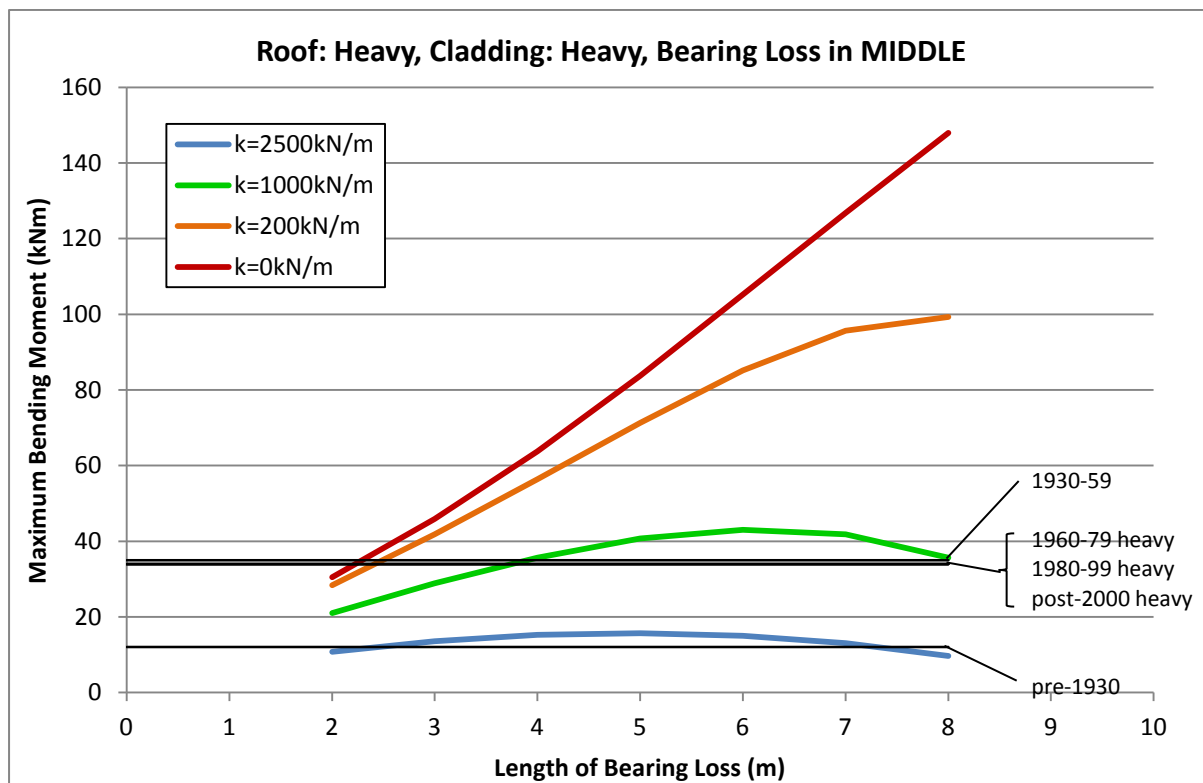


Figure A. 130. Moment demands for heavy roof and heavy cladding for different levels and lengths of bearing loss in centre. Yield capacities of the corresponding perimeter sections are also shown.

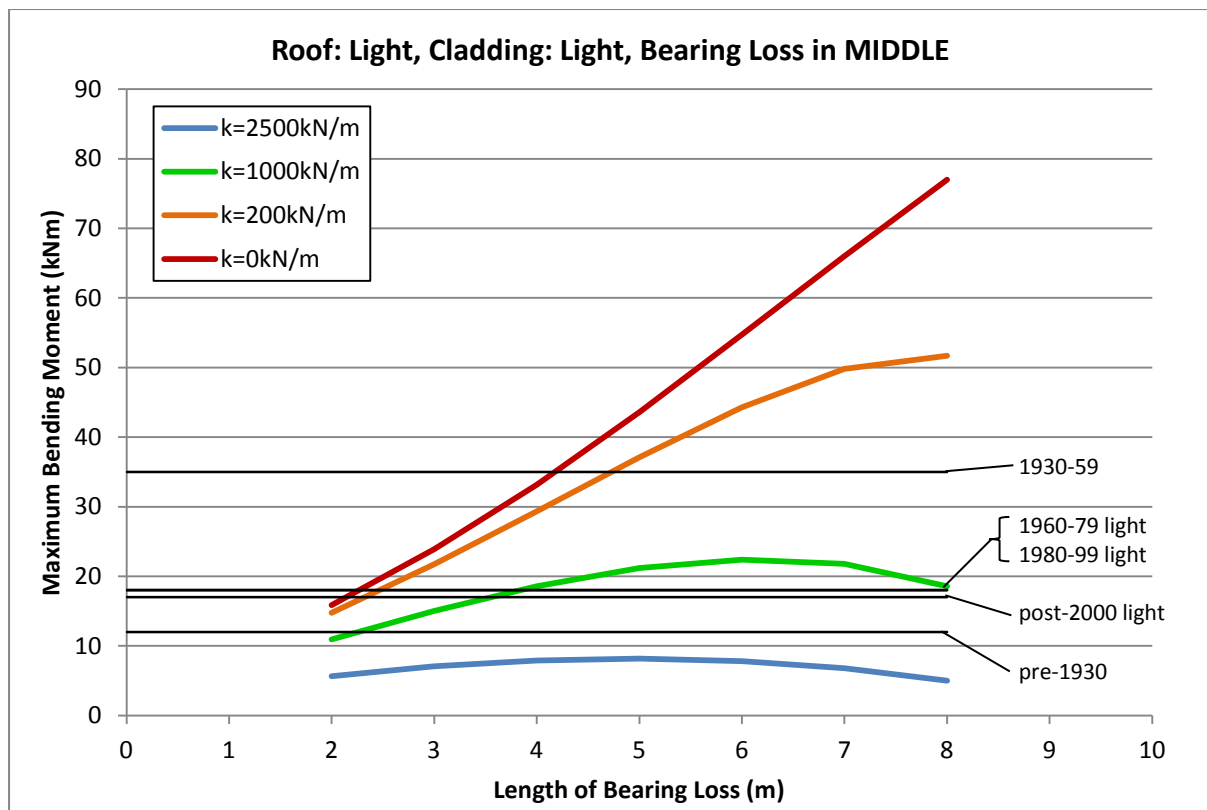


Figure A. 131. Moment demands for light roof and light cladding for different levels and lengths of bearing loss in centre. Yield capacities of the corresponding perimeter sections are also shown.

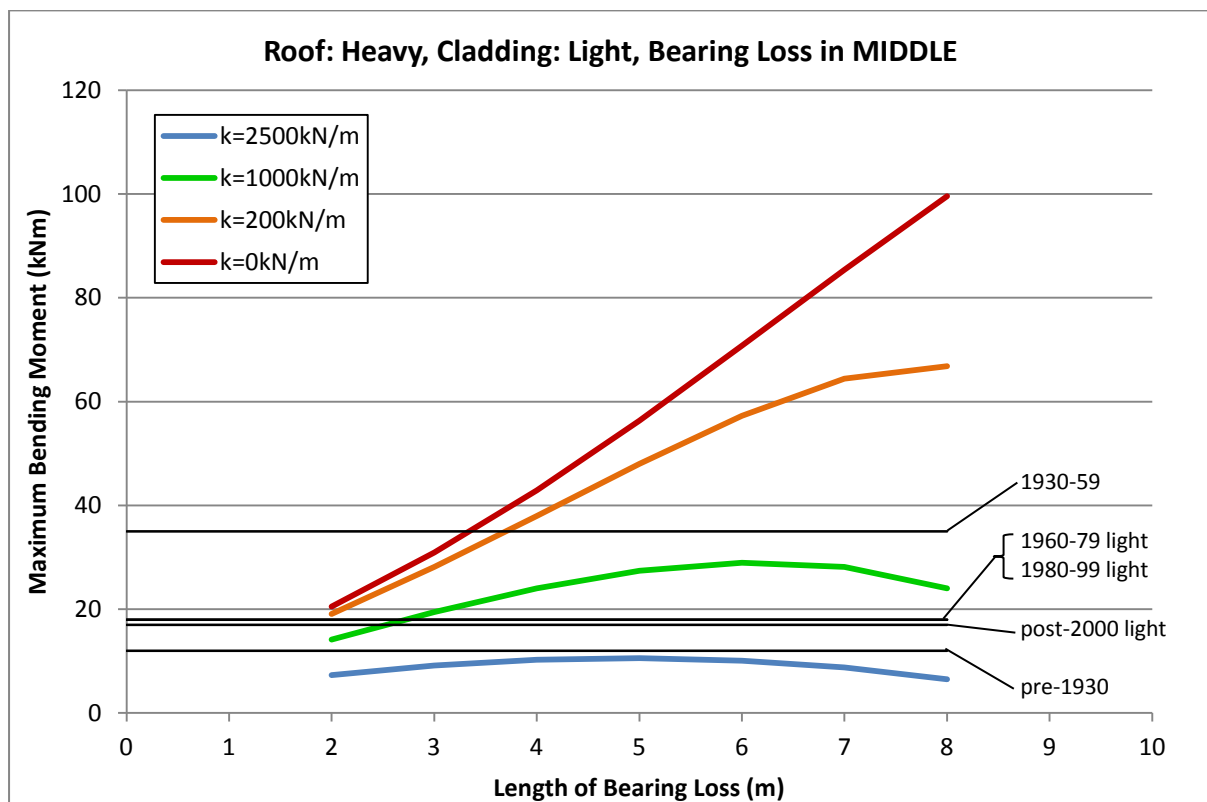


Figure A. 132. Moment demands for heavy roof and light cladding for different levels and lengths of bearing loss in centre. Yield capacities of the corresponding perimeter sections are also shown.

S.2: Bearing Loss at One End

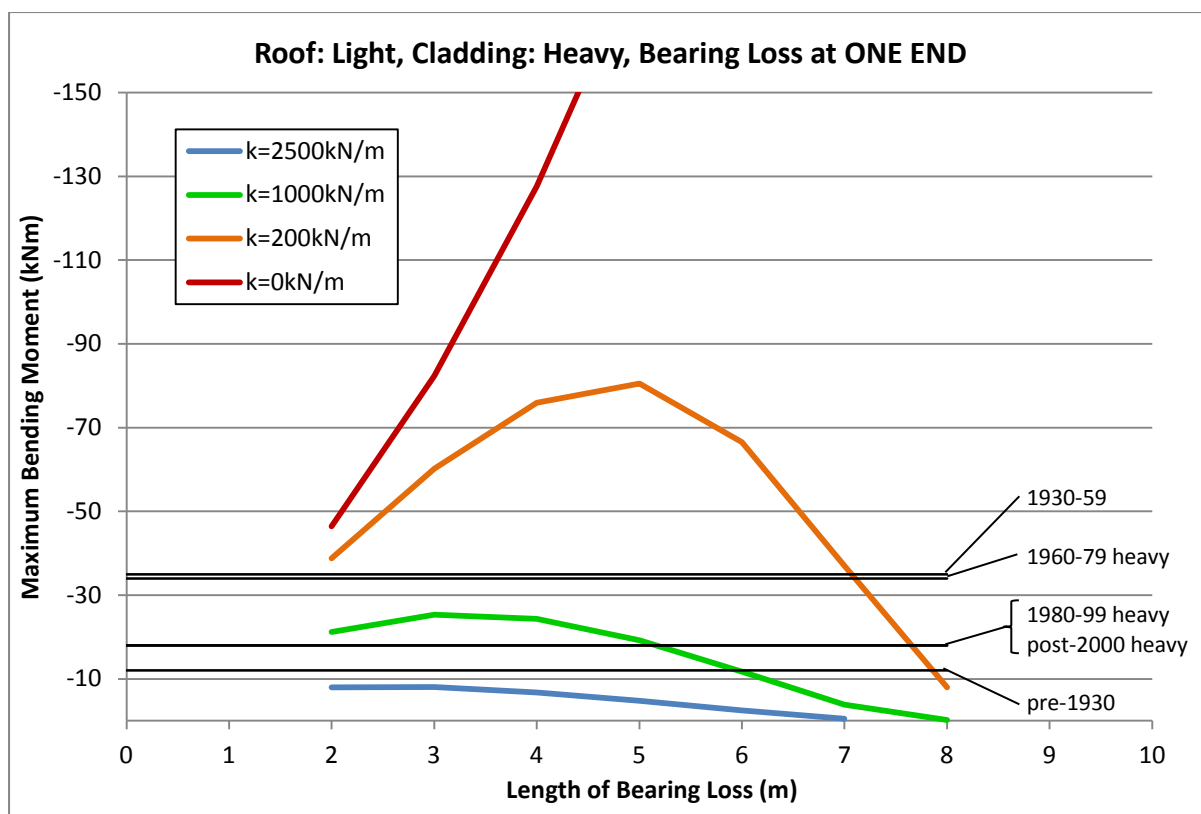


Figure A. 133. Moment demands for light roof and heavy cladding for different levels and lengths of bearing loss at one end. Yield capacities of the corresponding perimeter sections are also shown.

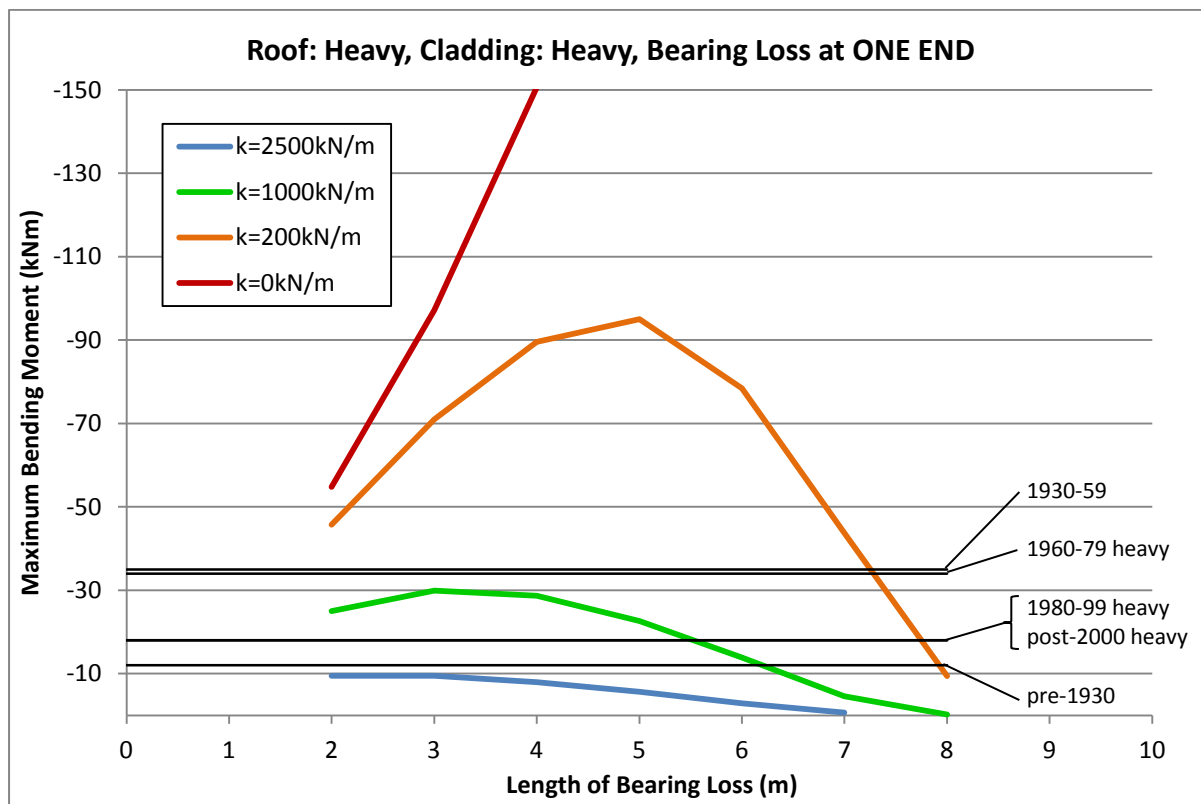


Figure A. 134. Moment demands for heavy roof and heavy cladding for different levels and lengths of bearing loss at one end. Yield capacities of the corresponding perimeter sections are also shown.

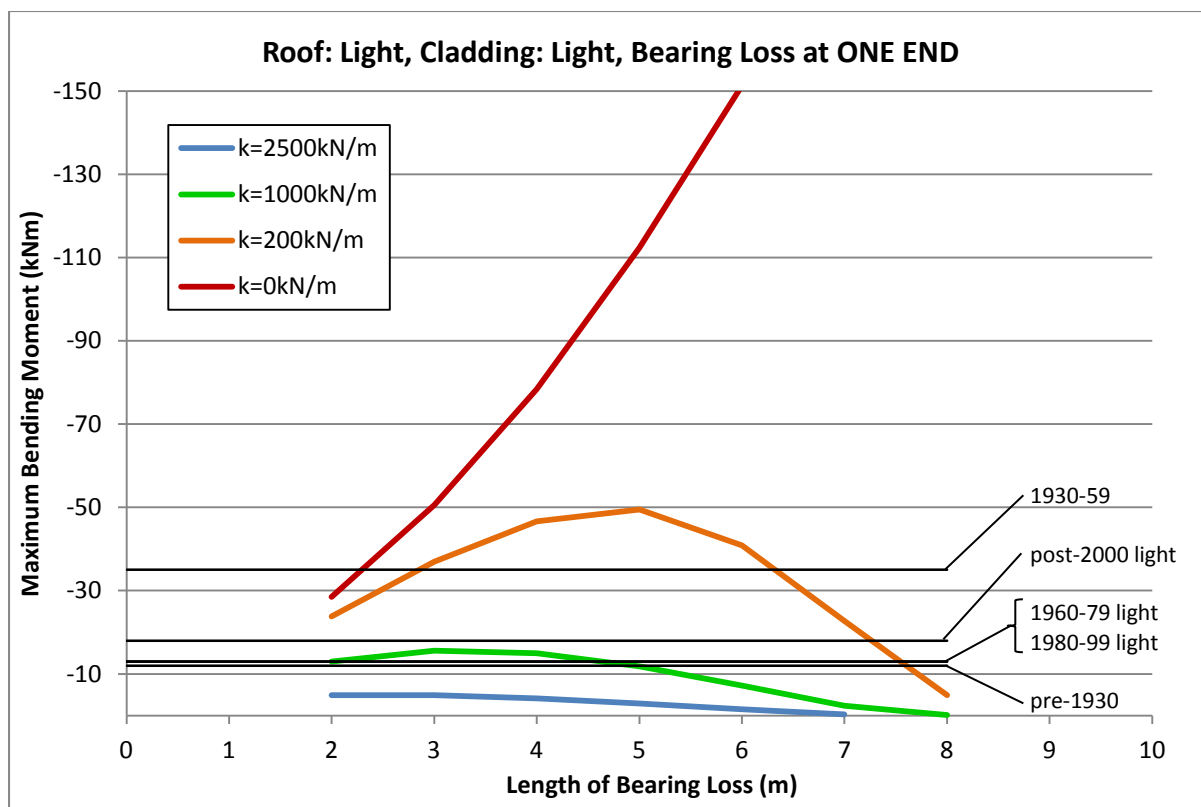


Figure A. 135. Moment demands for light roof and light cladding for different levels and lengths of bearing loss at one end. Yield capacities of the corresponding perimeter sections are also shown.

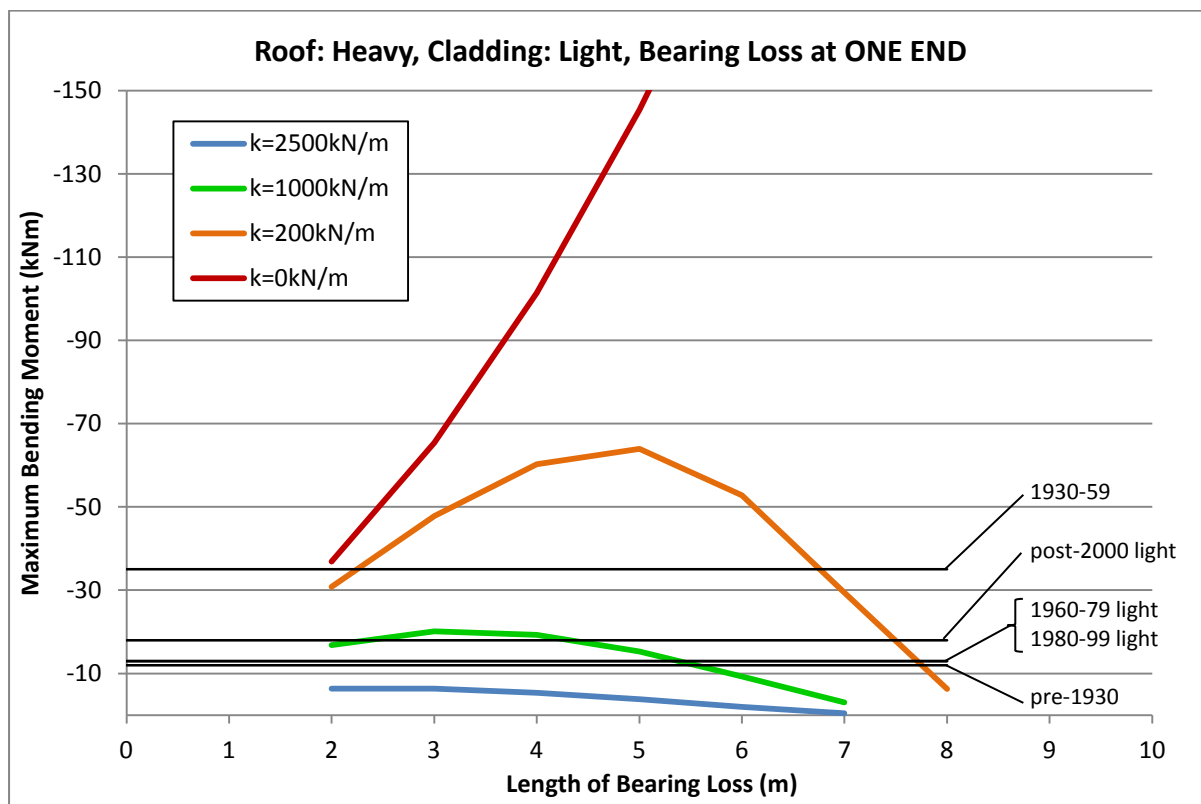


Figure A. 136. Moment demands for heavy roof and light cladding for different levels and lengths of bearing loss at one end. Yield capacities of the corresponding perimeter sections are also shown.

S.3: Bearing Loss at Both Ends

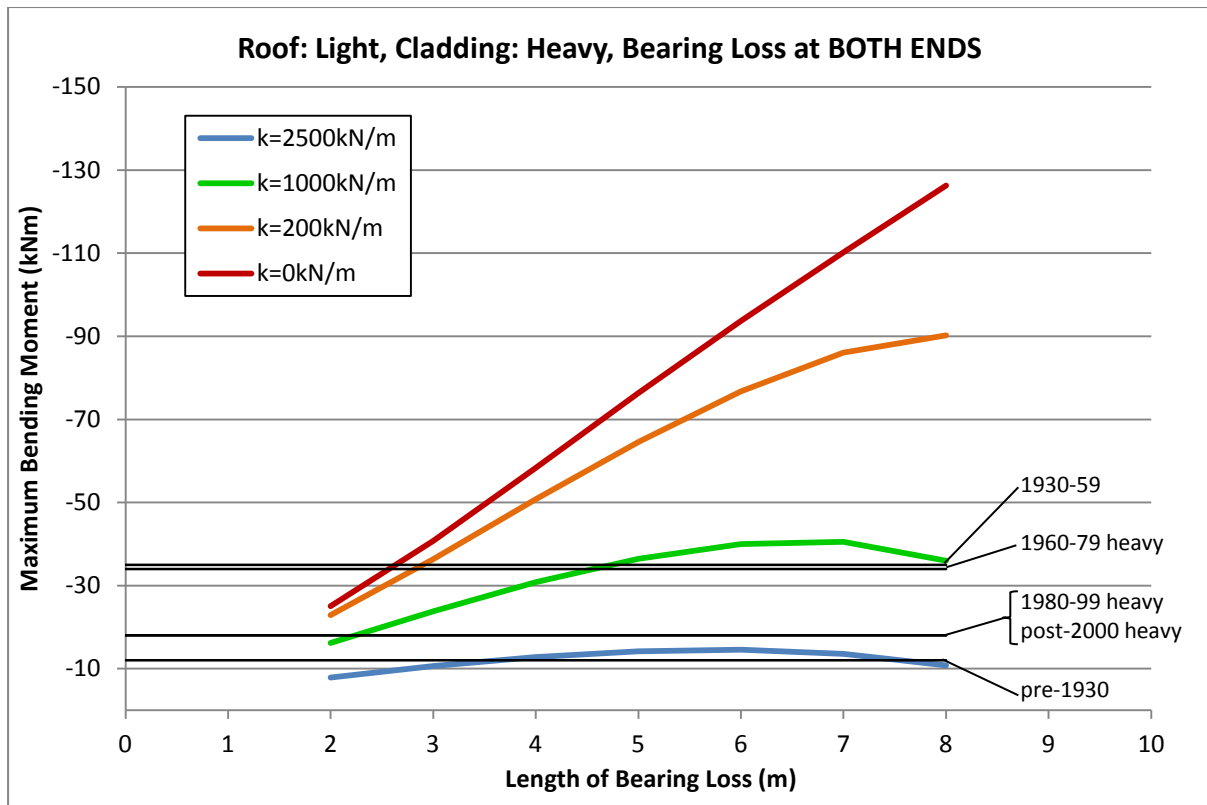


Figure A. 137. Moment demands for light roof and heavy cladding for different levels and lengths of bearing loss at both ends. Yield capacities of the corresponding perimeter sections are also shown.

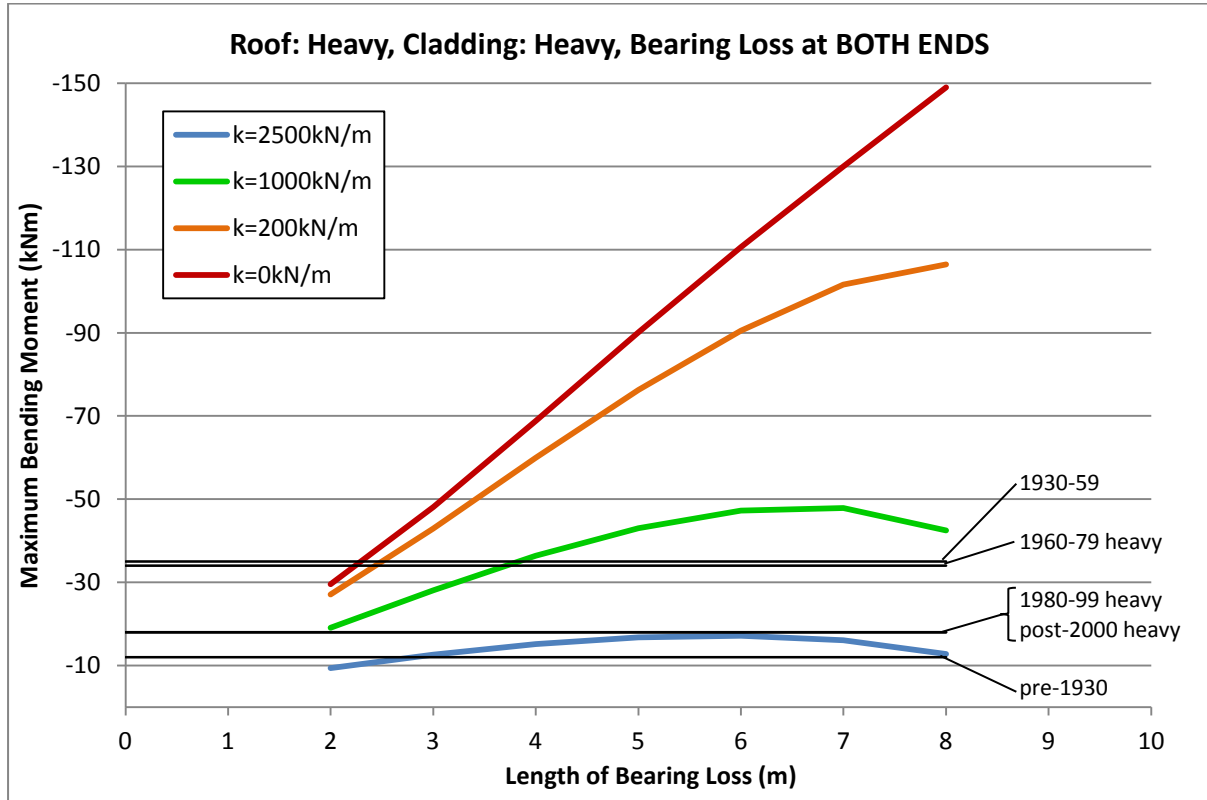


Figure A. 138. Moment demands for heavy roof and heavy cladding for different levels and lengths of bearing loss at both ends. Yield capacities of the corresponding perimeter sections are also shown.

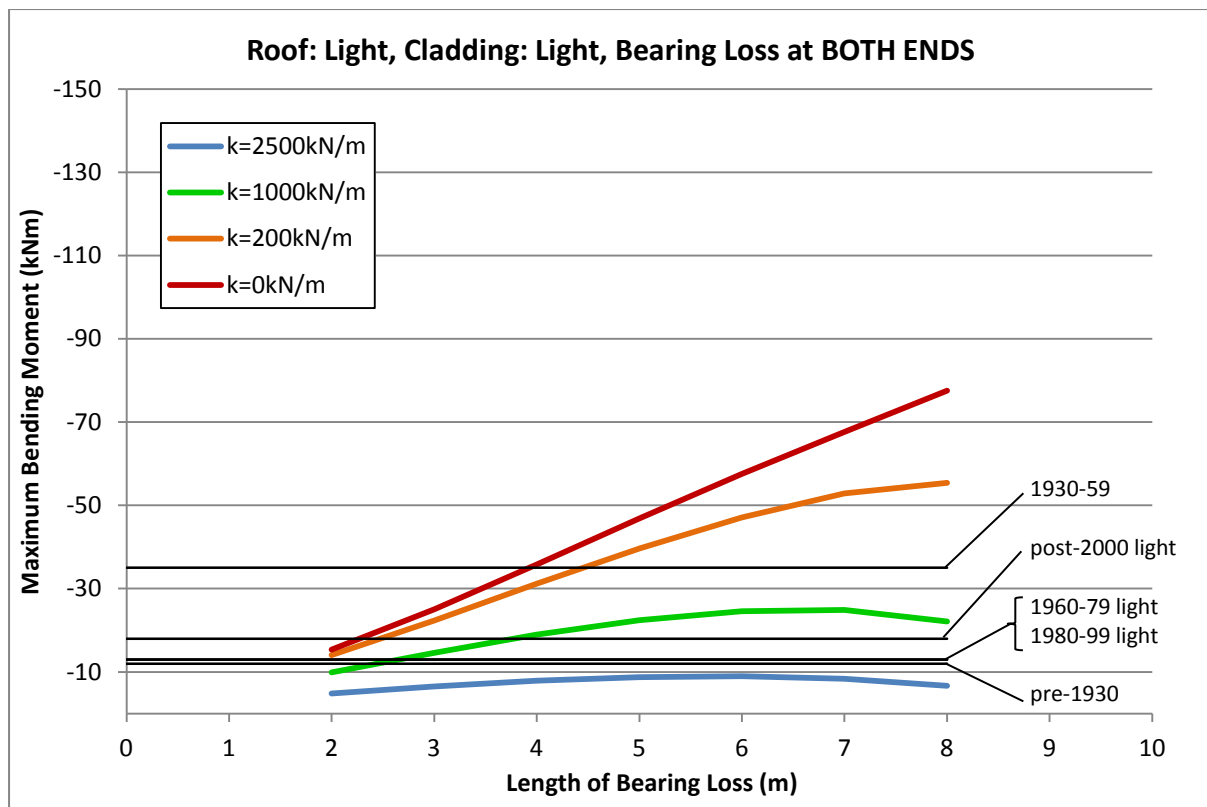


Figure A. 139. Moment demands for light roof and light cladding for different levels and lengths of bearing loss at both ends. Yield capacities of the corresponding perimeter sections are also shown.

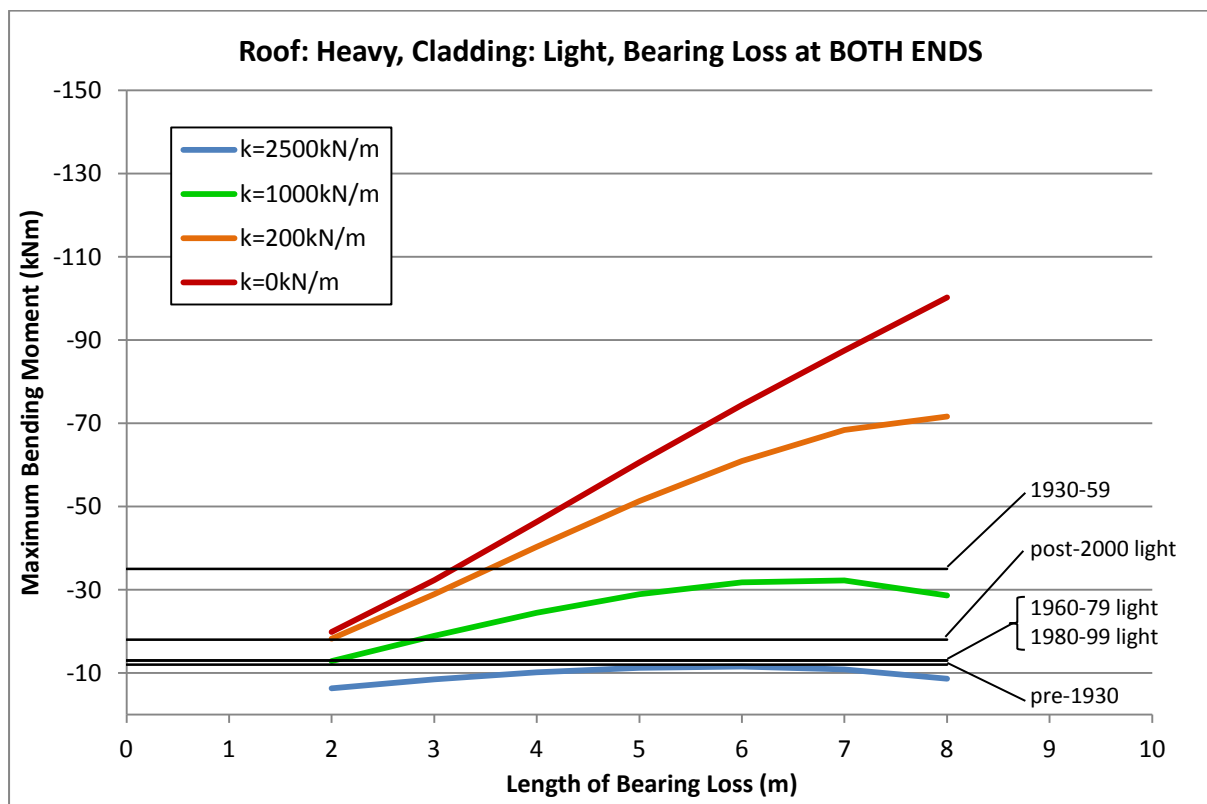


Figure A. 140. Moment demands for heavy roof and light cladding for different levels and lengths of bearing loss at both ends. Yield capacities of the corresponding perimeter sections are also shown.

Appendix T: Individual Housing Reports from Invasive Inspections

House ID:	House #1
Foundation Type:	Concrete Perimeter & ‘Piers’
Number of Stories:	1
Age:	1940 (1930-59)
Plan Area:	110
Plan Shape:	“L”
Cladding	Brick (solid)
Roof	Iron



Figure 1. Overview of property, showing direction of overall photo views.



Figure 2. Local suburb, showing distance to nearby water.

Earthquake	Liquefaction	Lateral Shift (LiDAR)	Settlement (LiDAR)
September 2010	High	0.3m NE	-0.1 to -0.4m
February 2011	High	0.1N	-0.1 to -0.3m
June 2011	Mod.	0.15m NW	0.1 to -0.1m
December 2011	-	0.1N	0.1 to -0.1m



Figure 3. P1: Looking West at the East (roadside) front of the house.



Figure 4. P2: Looking East at the Northwest corner of the house.



Figure 5. P3: Looking East at the West side of the house.

Foundation Details

Perimeter:

The perimeter is constructed with cast-in-place concrete. The aggregate is round stone, very uniformly graded between 10-20mm diameter. Large round stones of 200-300mm diameter have been used as fill throughout the perimeter of the house. The perimeter was quite weak around these large stones, and mostly broke up around them. The concrete mix was found to be dry in places, where gaps around aggregate were found (Figure 7). The perimeter extended about 500mm above the ground level, and 200-250mm into the ground, for a total height of 700-750mm. The widths were 200mm on N-S running walls and 250 on E-W running walls (Figure 6).

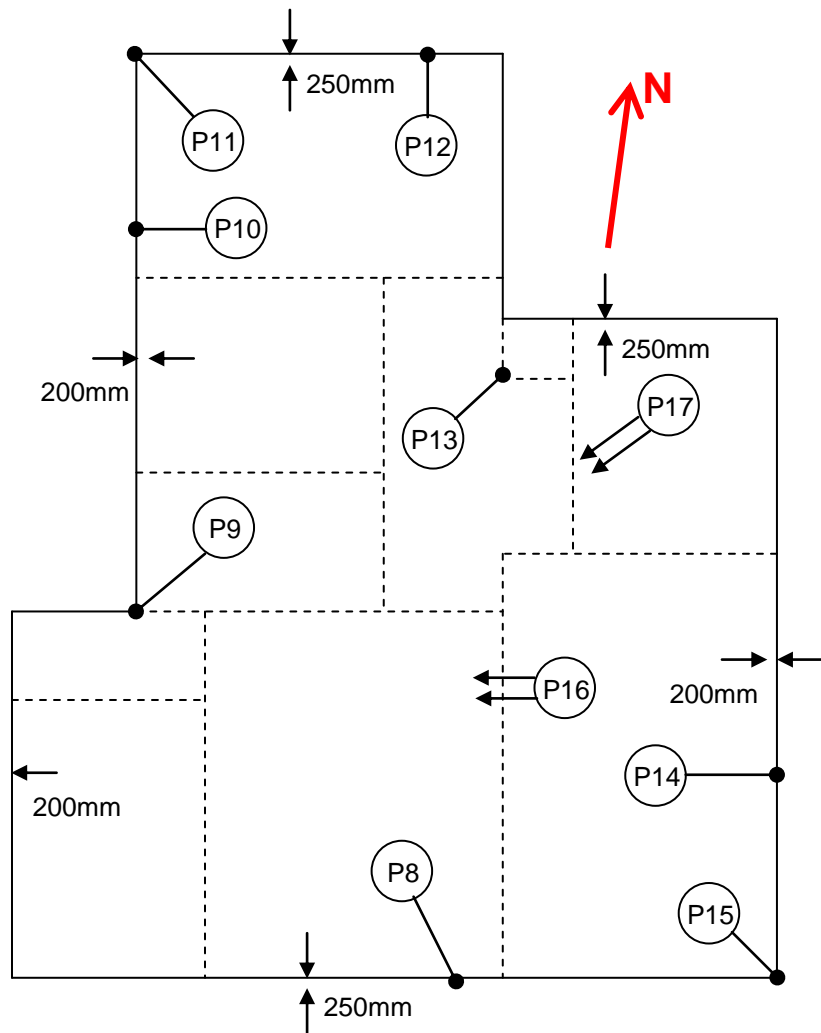


Figure 6. Floor plan showing general layout of house, with perimeter footing widths and locations of following photos.



Figure 7. Foundation details, (right) perimeter, showing large stones in concrete and (left) round aggregate in perimeter.

Piers:

The piers were round pre-cast concrete with a top diameter of 150mm and bottom diameter of ~200mm. They were set in the ground by ~100mm, giving ~400mm above ground. The footings for these were 200-400mm deep and ~300mm square/round cast-in-place again using the large stones as fill (as for the perimeter). The floor bearers were attached to the tops of these with wire legs (Figure 8).



Figure 8. Pier details, (left) pier in ground, (centre) pier and footing, (right) large stones in footing.

Ground:

Under the foundations and floor, the ground was pure sand. While there was liquefaction ejecta on top of the ground, and also mixed into the top layer, this pure sand extended fully to at least 1.8m, without any sign of soil or other materials (Figure 9).



Figure 9. 1.8m test pit under house, showing sand throughout depth.

Summary of Damage

Surroundings

The concrete deck on the West side of the house was tilting in different directions. It was separated from the perimeter foundation by ~10mm on its South side. There was a significant horizontal crack running the whole length on the N side right through, and a few minor cracks on top of the deck (Figure 10, P4).

There was damage to fences including leaning and broken parts, though it was not possible to tell if this was caused by the earthquake or not (Figure 10, P5).

There was significant damage to concrete paving, including extensive minor cracks in the driveway (again may be old expansion/construction related rather than earthquake) and significant heaving at the street end of the driveway caused by sand ejecta (Figure 10, P6).

There were small amounts of sand ejecta left on the section, but most had been removed or was overgrown (Figure 10, P7).



P4



P5



P6



P7

Figure 10. P4: Cracking along concrete deck. P5: Damage to fence. P6: Heaving of pavement in driveway. P7: Some small amounts of sand ejecta left in back lawn.

Structural Damage

There was significant structural damage over the whole building. The exterior cladding had numerous diagonal cracks in the brickwork, from 1-10mm, with a smaller number of horizontal and vertical cracks too (Figure 11 and Figure 12). The majority of these cracks caused horizontal separation between the bricks, suggesting foundation movement had caused bits of the house to tilt differentially and pull away from each other. There were two significant cracks on either side of the Northwest corner of the house. The one on the West wall was diagonal and extended the full height of the wall, 0mm in width at the bottom (close to the corner) to 10mm of horizontal separation at the top away from the corner (Figure 12, P10). On the North wall, another diagonal crack started with 0mm at the bottom at the corner, and opened to 10mm horizontal separation at the bottom window corner (Figure 12, P11). This was then continued as a 10-15mm horizontal separation between the window jam and bricks at the opposite (top) corner of the window (Figure 12, P12). This suggests that this whole corner of the house is separating out away from the rest of the house as the rest of the house tilts away from it.

Another significant area of damage was in the middle of the East wall. Here, the top four layers of bricks between two windows were pushed back into the house by up to 30mm. This may have been caused by the whole rest of the wall pushing out, or by shaking damage dislodging this section of bricks (Figure 12, P14).

There were quite a number of doors and windows that were sticking or wouldn't close properly, and a couple of doors that swung closed on their own. Damage to the interior was generally minor, with small cracks in the plaster at wall/wall and wall/ceiling interfaces and above windows and doors.



P8



P9

Figure 11. P8: Diagonal crack in brickwork showing horizontal separation of 5mm. P9: Separation between door frame and brick cladding.



P10



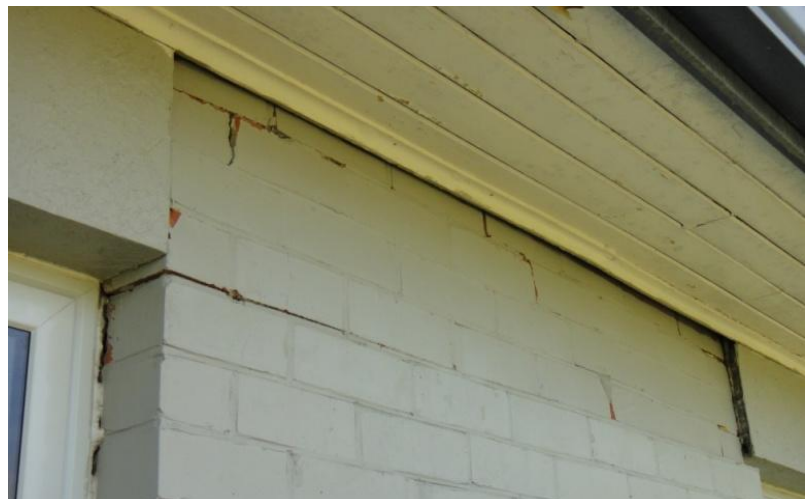
P11



P12



P13



P14

Figure 12. P10: Diagonal crack 0-10mm horizontal separation from bottom of wall to top. P11: Diagonal crack causing 10mm horizontal separation and (P12) 15mm separation above window. P13: Cracking down front door frame. P14: Push out of wall/in of top bricks by 10-30mm.

Foundation Damage

There was significant damage to the foundation throughout the house. Damage to the perimeter however was not high. There were a total of 5 cracks around the perimeter, all of them 1-2mm wide (Figure 13, P15).

However, the floor was very severely tilted. From the high point at the North corner of the East wall, there was 150mm total differential settlement to the Southwest corner. Although sloping of the floor was relatively consistent in this direction (to the Southwest), there were some variations in slope angles that showed room humping was present. There was one area in the kitchen where humping could be felt underfoot, and other more minor areas were identified through the floor slope angles. When the floor was lifted, extensive liquefaction was discovered under the floor covering the Southwest half of the house, along an approximate NW-SE line along the middle of the plan area. This explains the high level of tilt, and the floor humping present.



P15



P16



P17

Figure 13. P15: Minor cracking in perimeter. P16: Large sand boils under foundation. P17: Looking SW over foundation showing extensive sand ejecta.

Overall Building Deformation

The building had an overall differential settlement of 150mm to the Southwest with a maximum slope angle of 1.3° to the West. The house tilting was quite uniform overall, but there was some local room humping, in the Southwest rooms and the kitchen and in the North bedroom being the main areas. Extensive sand ejecta from liquefaction was found under half the house, and under all areas that suffered local humping (Figure 13). The sand ejecta was located under the Southwest half of the house, and was found to be consistent throughout the first 2m of a test pit that was dug. This is assumed to be the reason for the house tilting to the Southwest, as that is where the most severe liquefaction occurred.

The concrete perimeter suffered only minor cracking in five locations around the perimeter, despite the large settlement and the weak construction of the perimeter, with no reinforcing, round aggregate and large stones for fill. This is due to the relatively uniform nature (around the perimeter) of the settlement, and possibly that the whole building was built on uniform sandy material, which provided little resistance to the settlement of the house during liquefaction.

Overall, the superstructure was found to be tilting within $\sim 0.2^\circ$ of the floor slopes adjacent, except in the kitchen, and the North bedroom. In the kitchen this was most likely due to the floor humping giving different slope angles on the floor from those which represented the overall tilt of the house at that location. In the North bedroom, the walls in the Northwest corner of this room were tilting out from the rest of the house towards the Northwest. This was shown by long diagonal cracks in the brick cladding in both walls outside, which went from 0mm width at the bottom, close to the Northwest corner, to 10-15mm horizontal separation at the top of the wall, away from the corner.

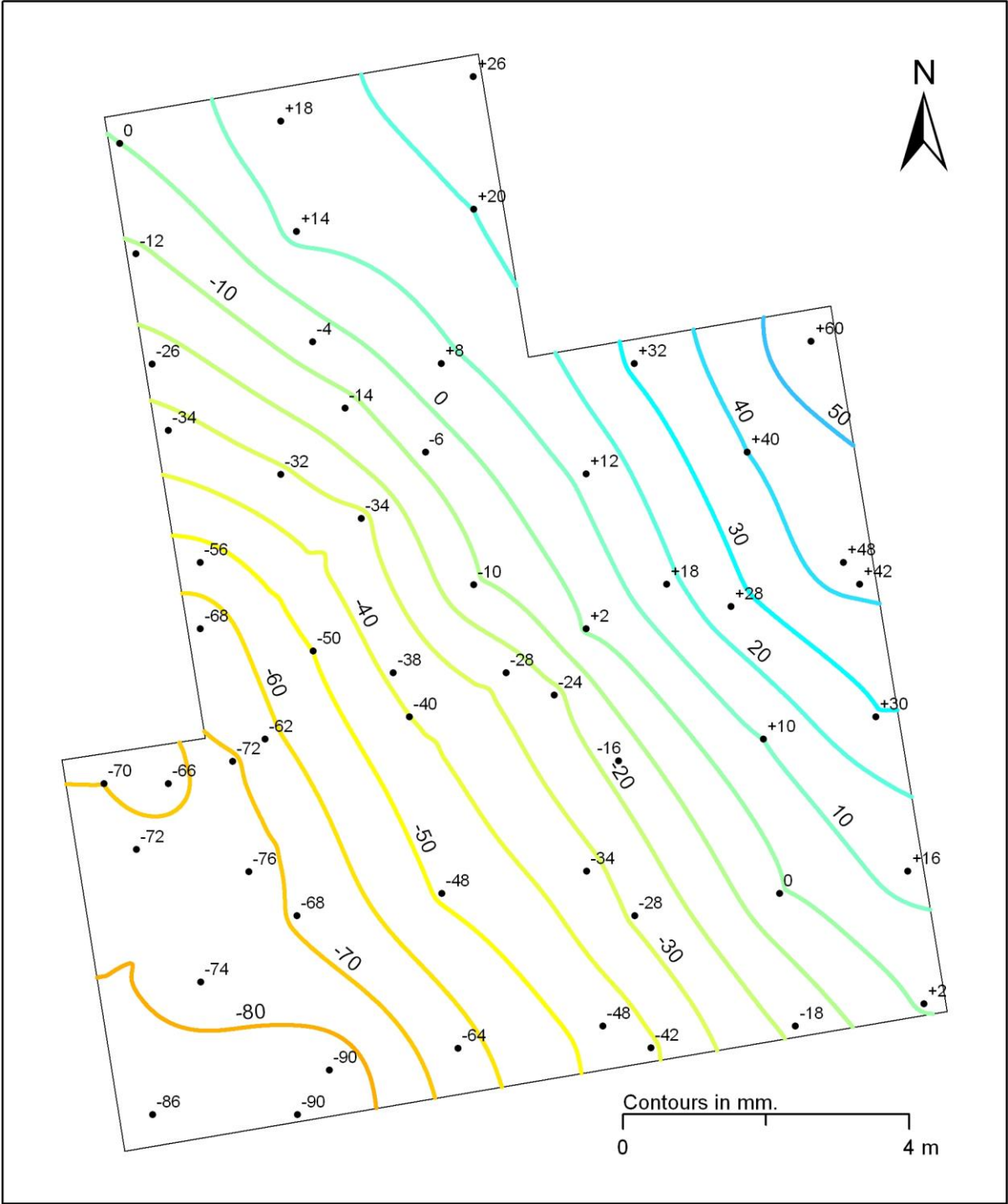
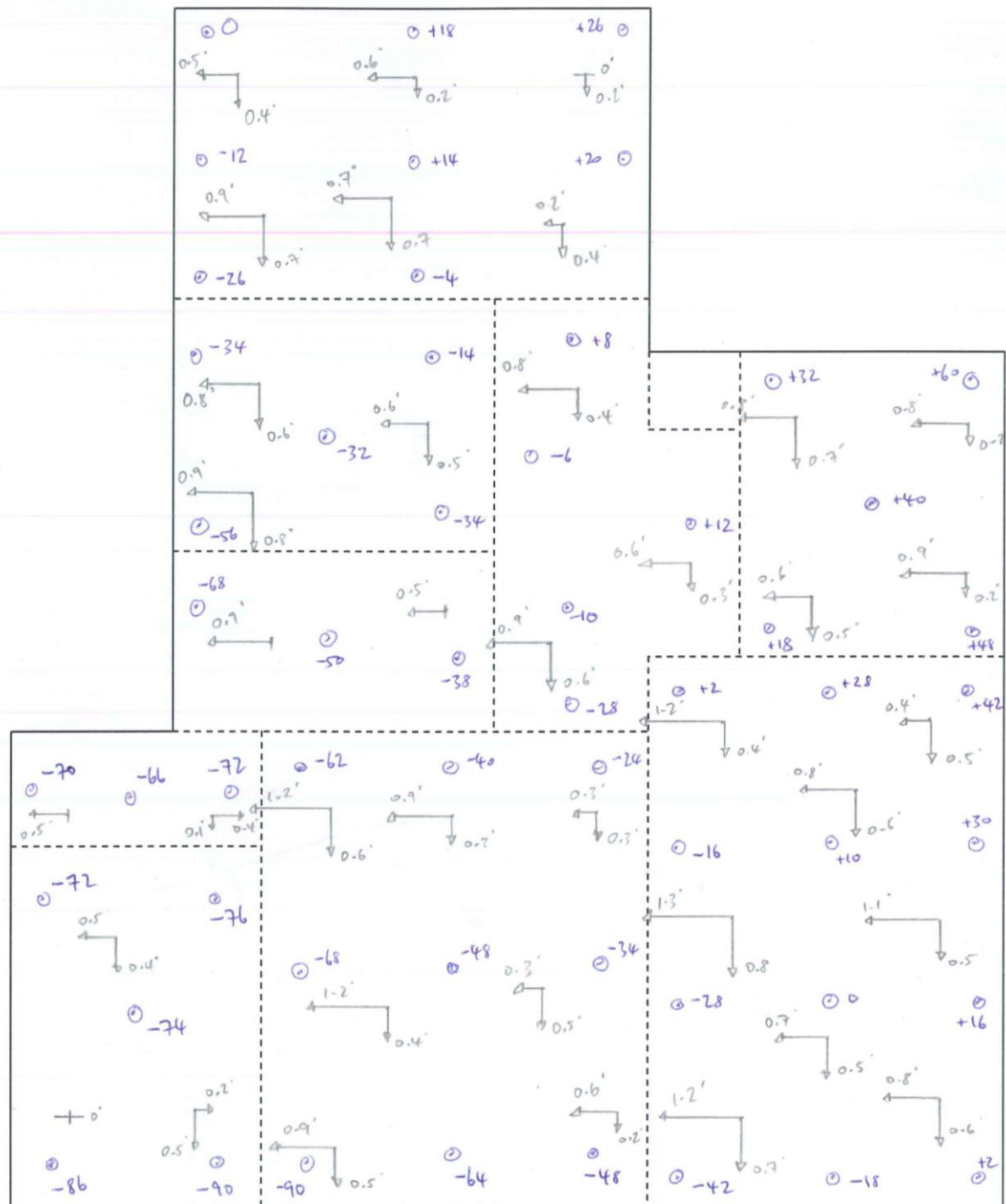


Figure 14. Floor plan showing floor levels and contours.



House ID:	House #2
Foundation Type:	Perimeter
Number of Stories:	1
Age:	1950 (1930-59)
Plan Area:	115m ²
Plan Shape:	"T"
Cladding	Concrete Block with Plaster
Roof	Heavy Tiles



Figure 1. Overview of property, showing direction of overall photo views.



Figure 2. Local suburb, showing distance to nearby water.

Earthquake	Liquefaction	Lateral Shift (LiDAR)	Settlement (LiDAR)
September 2010	-	0.1m NW	-0.1 to -0.3m
February 2011	-	0.3m W	-0.1 to -0.3m
June 2011	-	0.1m NW	0 to -0.2m
December 2011	-	0.1m N	0.1 to -0.1m



Figure 3. P1: View of the East side of the house, looking SW.



Figure 4. P2: Southeast corner of the house, looking Northwest.



Figure 5. P3: West side of the house, looking East.

Foundation Details

There are three separate parts to the foundation structure. There is a low slab-on-grade patio on the West side. The main body of the house with one type of perimeter wall and piers, and one small extension on each side with slightly different perimeter wall and pier arrangements (Figure 6).

Perimeter:

The whole perimeter is constructed of cast-in-place concrete. The aggregate is mostly round stone with some angled aggregate and reasonably graded between 2-20mm in diameter.

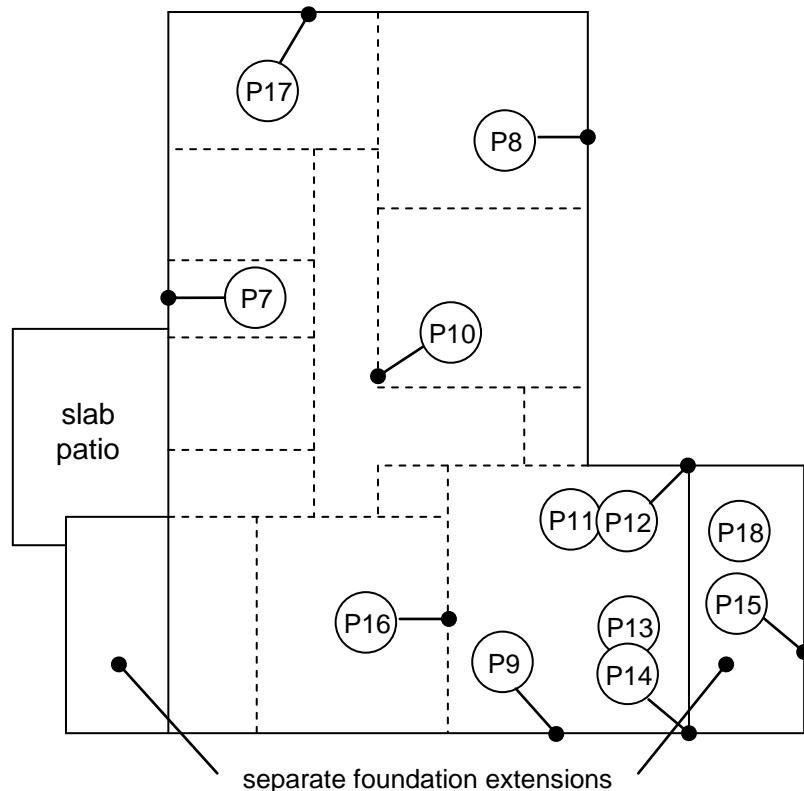


Figure 6. Floor plan showing general layout of house, foundation details and locations of following photos.

Main Perimeter:

- 600mm total depth. 300-400mm out of ground, 200-300mm in ground.
- Un-boxed base (100-150mm deep) up to ~50mm wider than boxed perimeter.
- 230-240mm wide at top all around.
- 2x R16's placed 50-100mm from bottom longitudinally.
- R8 every ~1.5m going into timber base of walls.

Extension Perimeters:

- 450-500mm deep, 300mm out of ground, 100-150mm of unboxed, no wider.
- ~250mm wide, 2x R12's placed 50-150mm from bottom.

There was no physical connection between the original and extended perimeter walls, they simply sat beside each other. Both perimeter types came out of the ground quite easily, as they were not very deep, though took some breaking up. There were areas on both perimeters where there was no mix around the aggregate, and there were just stones touching each other (Figure 7).



Figure 7. Perimeter details, (left) main perimeter out of ground, (centre) dry area of concrete, (right) no connection between new and old perimeter.

Piers:

The main part of the foundation had 150-170mm square, cast-in-place piers ~350mm high above the ground. Their footings were ~300-350mm square and ~200-250mm deep. As for the perimeter, the aggregate was quite good, with a good mix of round and angular stones of varying sizes. However the concrete was much too dry, with number of piers having significant areas at the top with no mix around the aggregate (Figure 8).

The piers in the extension were standard round pre-cast concrete with a top diameter of 150mm and bottom diameter of ~200mm. They were the same height, around 300-350mm above the ground. The floor bearers were attached to the tops of all piers with wire legs.



Figure 8. Pier details, (left) square cast-in-place pier, (centre) close-up of dry concrete, (right) round pre-cast piers.

Ground:

The only sand ejecta under the foundation was found under the West extension. Here the sand was very thick, 2/3 filling the extension at up to ~250mm deep. This extension was under the kitchen, so a high concentration of piping in this area may have led to the severely localised ejecta (Figure 9).



Figure 9. P4: Extension under kitchen, showing large volumes of sand ejecta.

The ground under the house was all soil, becoming quite cohesive and clayey below ~500mm to ~1.7m. This material had very low plasticity however. Below 1.7m, to the depth of the hole at ~2.2m was grey sandy-silty material, which was quite wet, and when vibrated in the hand, became wet on top. This is likely the material that liquefied, causing the settlement of the house. And the 1.7m thick clayey crust layer is the likely reason that little surface liquefaction was found under the house (Figure 10).



Figure 10. Ground details, (left) clayey soil down to ~1.7m and (right) sandy silty material turning wet in the hand.

Summary of Damage

Surroundings

There was a high level of heaving and settlement of the various concrete paving. There was also plenty of cracking in this paving, however it was not possible to tell if this was historic or due to the earthquake.

There were signs of liquefaction ejecta still present on the property, along the front boundary in the footpath, and up to 100-150mm thick in the garage. However there was not a lot on the lawn itself (Figure 11).

There was some damage to fences, with low levels of leaning and some gaps opened up.



P5



P6

Figure 11. P5: Sand ejecta in footpath at front. P6: Sand ejecta inside garage.

Structural Damage

Structural damage to the building ranged from low to high, with all the serious damage being concentrated in one room.

The main part of the house (North section) suffered low to moderate damage. On the West and North sides there were sections where the wall and underside of the roof were separated by 1-5mm (Figure 12). On the East side there were two cases of horizontal separation between walls and a window and the front door by 1-5mm. There were also minor vertical cracks in the stucco all the way around from 0-5mm. The interior of this section also suffered minor damage, such as hairline-1mm cracks in the plaster above window and door corners with one hairline crack in the ceiling of the bathroom, with a small crack in the tiling underneath (Figure 13). In this section of the house the doors were moving ok.

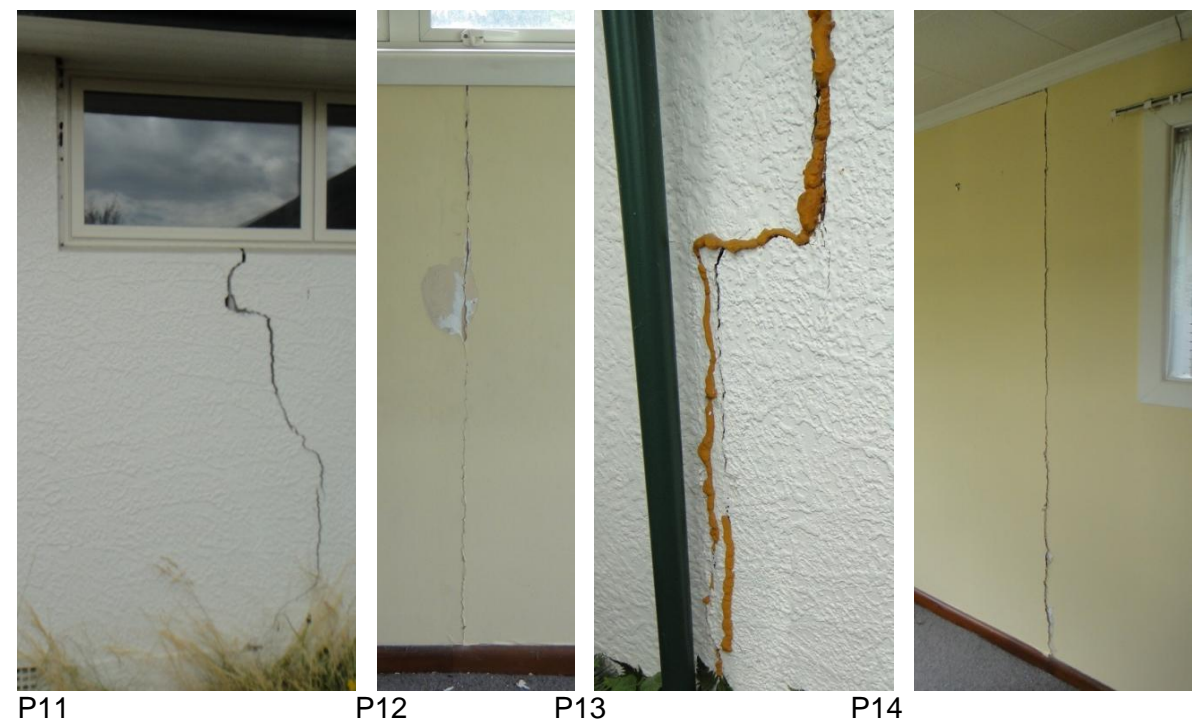
The worst damaged part of the house was the South and East section. On both the North and South walls of the Southeast room of the house, there were large matching structural cracks running through the stucco, and showing separation between the concrete blocks beneath. On the North wall, the crack was 2mm wide at the bottom, and cracked to 20mm width beneath the high window. From there it was translated to a separation between the window and wall of 20-30mm to the top of the wall (Figure 14, P11 and P12). On the South wall there were two cracks which opened up from 0mm at the bottom to 10mm at the top right beside each other. It is assumed these occurred in subsequent earthquakes as one was filled (Figure 14, P13 and P14). On the East wall of this section, at both North and South corners, there are cracks running up the wall and along beside the window from 1mm at the base to 5mm at the top as the walls either side of the window push out. Inside this room, the external cracks were matched by significant vertical cracks in the plasterboard at joints. There was also some racking offset between sections of plasterboard. In this South section of the house there were two door frames that were badly racked, preventing the doors from being closed (Figure 14).



Figure 12. Minor exterior damage, separation between wall/roof (P7) and wall/window (P8).



P9 P10
Figure 13. Minor interior damage: Joint crack in ceiling (P9) and cracking above door in plaster (P10).



P15 P16
Figure 14. High structural damage: Two matching exterior (P11, P13) and interior cracks (P12, P14). P15: High separation between wall and window. P16: Racking of door frame.

Foundation Damage

Foundation damage was relatively low, despite the high levels of differential settlement and tilting of the building.

Eight hairline-1mm cracks were found around the concrete perimeter, which is a small number (Figure 15). The one 2mm crack identified was below the large structural crack going the height of the wall on the North wall of the Southeast room. This suggests that the perimeter was constructed well (good aggregate and reinforcing in the bottom), despite being a little dry in places.

There was some minor humping in a couple of the rooms, namely the Northeast bedroom and the middle rooms on the South side. However this humping was minor; no sand ejecta was found under these, and overall the sloping in the floor was uniform, though in different directions.



Figure 15. P17: Hairline crack in perimeter.

Overall Building Deformation

The building suffered complex overall movement, caused by a number of factors. Ground conditions under the building were better than have been observed at other sites, resulting in overall differential settlement of 70mm and low damage to the reasonably well-constructed perimeter foundation wall.

The main body of the house settled 70mm overall from the Northeast corner to the far Southwest corner quite consistently, with some minor room humping in between.

The two rooms in the South and East part of the house had more complex behaviour. Between the West and East sides there was overall hogging of the floor by ~50mm in the middle, with low room humping in both rooms near the centre. No liquefaction ejecta was found under these locations that may have caused the humping, so it was assumed to be caused by settlement of the walls. This hogging and humping caused significant racking of two door frames (Figure 14, P16). The protruding Southeast room of the house had severe tilting to the East by up to 1.3°, tilting in three somewhat separate sections, rather than smooth curved tilt. The East ~2m of the room was tilting the most, with a differential floor slope between this and the adjacent section of 0.4-0.9°. This caused the large external structural cracks in both North and South walls (Figure 14, P11-P14) to open up towards the roof as it tilted away from the rest of the house, and caused the pushing out of the end (East) wall around the top window. This movement also caused joint cracking and racking of sections of plasterboard in the end section.

Upon demolition of the house, it was found that this end section was actually an extension to the main house. The perimeter foundation wall was cast up against the old wall, but had no physical attachment (Figure 16). This, combined with the heavy cladding and roofing material, allowed this to break off and settle more than the rest of the house which was better connected together, and hence caused the only significant structural crack in the house, and crack (actually separation) in the perimeter. The old East perimeter wall was no longer carrying any significant load, as this was all transferred to the new perimeter, but was still able to provide significant stiffness to the rest of the main body of the floor, reducing settlement in the original part of the house.



Figure 16. P18: East end section of the house showing separate perimeter.

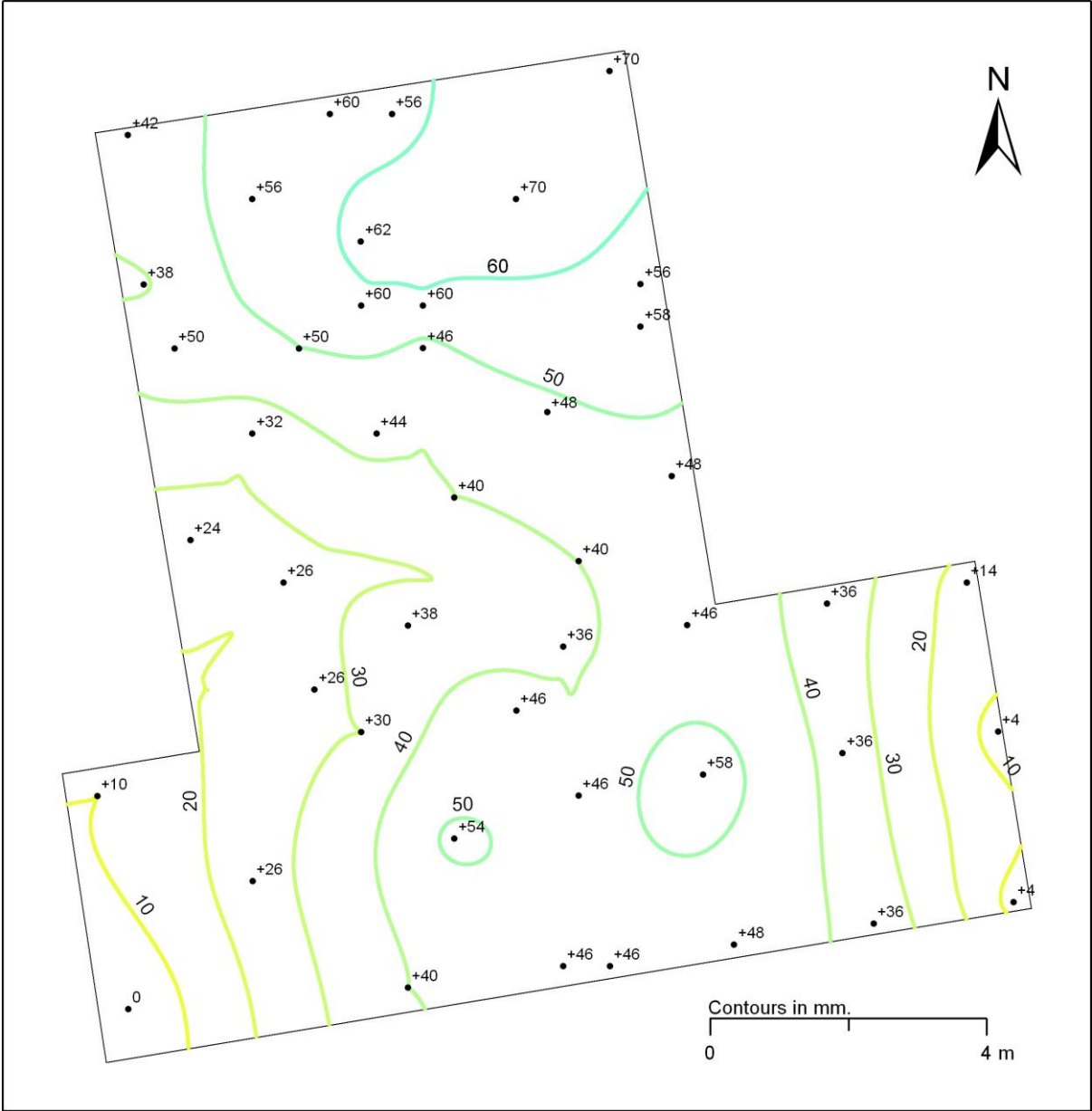


Figure 17. Floor plan showing floor levels and contours.

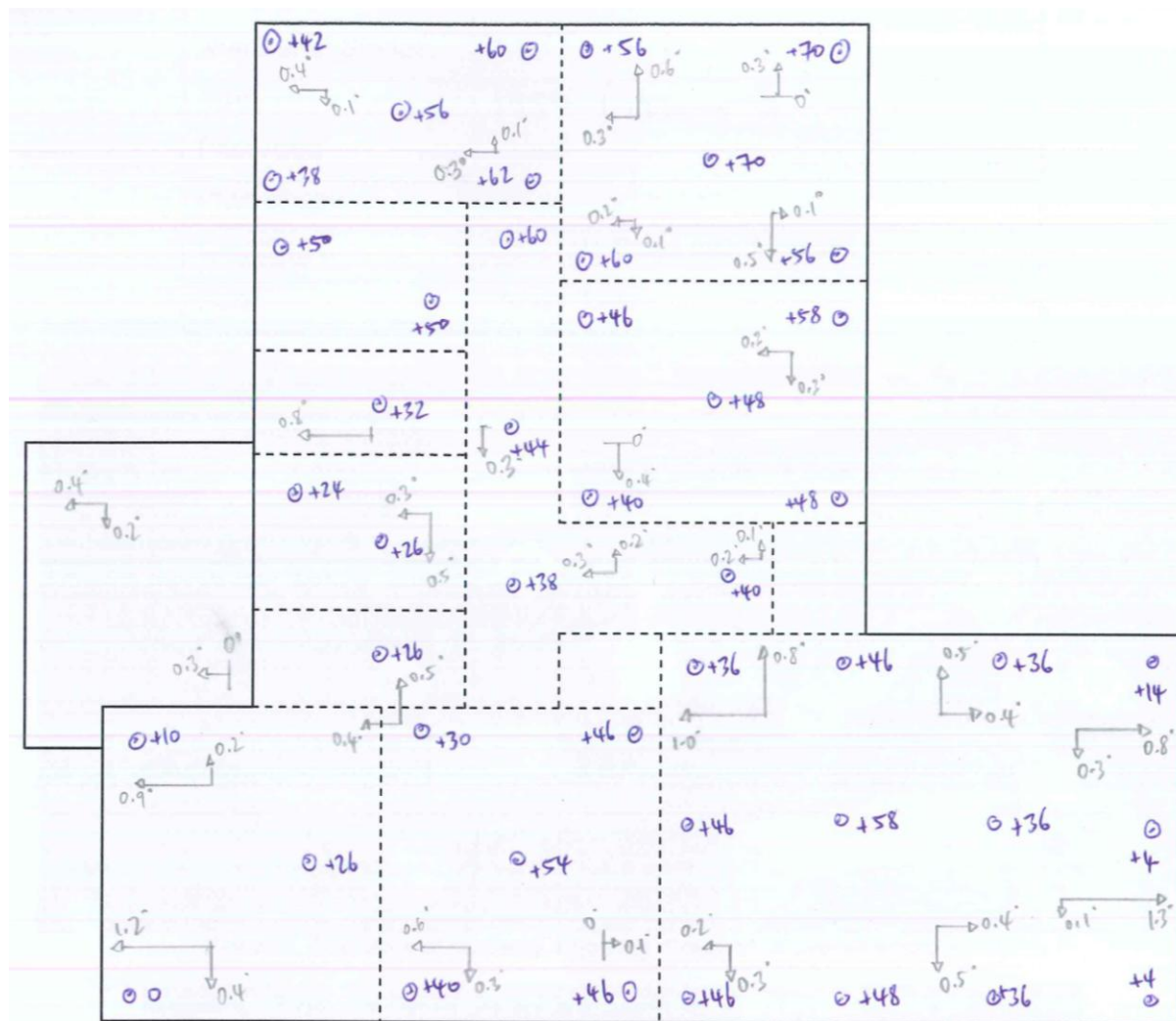


Figure 18. House plan layout showing room locations and floor slopes.

House ID:	House #3
Foundation Type:	Perimeter
Number of Stories:	1
Age:	1940 (1930-59)
Plan Area:	80m ²
Plan Shape:	Rectangle
Cladding	Stucco
Roof	Metal tiles



Figure 1. Overview of property, showing direction of overall photo views.



Figure 2. Local suburb, showing distance to nearby water.

Earthquake	Liquefaction	Lateral Shift (LiDAR)	Settlement (LiDAR)
September 2010	High	1m SE	0.1 to -0.1m
February 2011	Severe	0.4m S	-0.1 to -0.3m
June 2011	High	0.1m SW	-0.1 to -0.2m
December 2011	Low	-	0.1 to -0.1m



Figure 3. P1: West side of house.



P2



P3

Figure 4. P2: North side of house. P3: South side of house.



Figure 5. P4: East side of house.

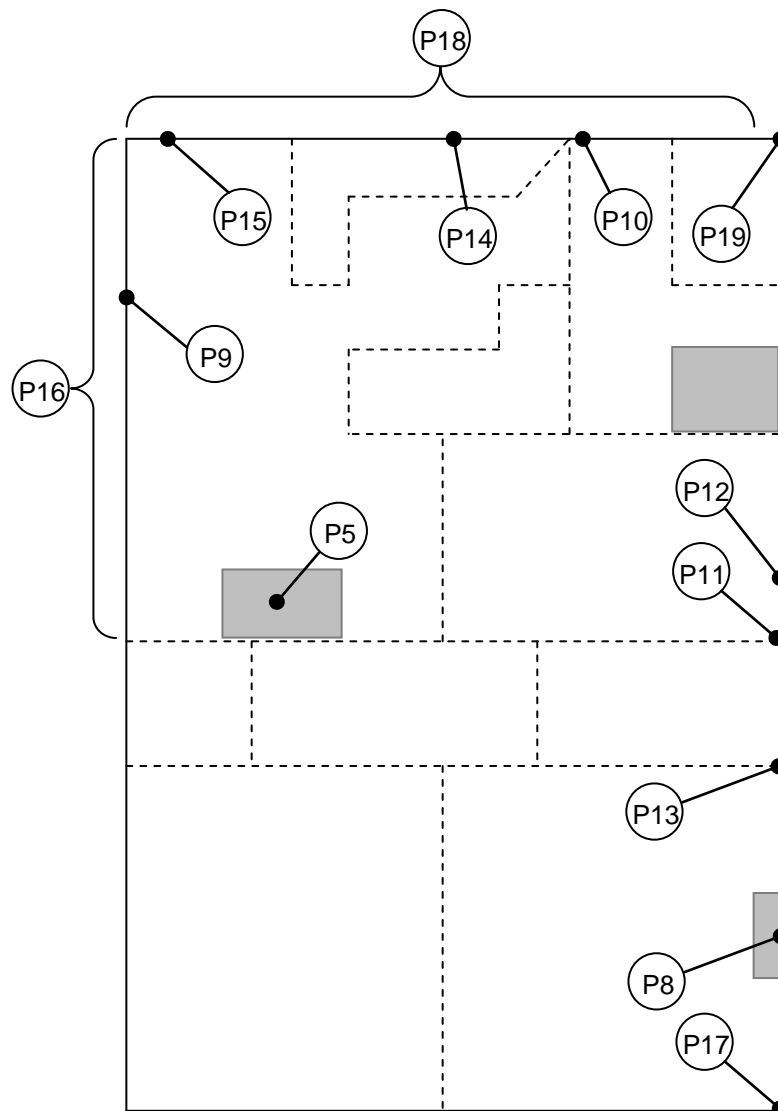
Foundation Details

Figure 6. Floor plan showing general layout of house, with locations of following photos.

There were three large cast-in-situ concrete footings, shown on the floor plan in Figure 6, and in Figure 7 below. These were assumed to be for old fire places and steps.



Figure 7. P5: Large cast-in-situ concrete footing within floor plan.

Perimeter:

The perimeter was constructed of cast-in-place concrete and was not reinforced. The aggregate was uniformly graded, round stone. The concrete mix was very dry, with lots of aggregate with large gaps around it, particularly on the inside of the perimeter wall. Large stones were used as fill in the bottom of the perimeter wall, up to 200-300mm in diameter. The perimeter broke smoothly and easily around these stones when it was lifted up (Figure 8).

The perimeter wall was 130mm wide all around and 600mm deep, with about 350mm out of the ground, and 250mm in ground, which varied if measured inside or outside the footing.



Figure 8. Perimeter details, (left) dry concrete mix showing gaps around aggregate and (right) large stones cast into perimeter as fill, breaking apart cleanly.

Piers:

The piers were square, precast concrete. They measured 180x180mm at the bottom, and tapered to 140x140mm at the top. They were 450-600mm long and cast into the ground between 0-250mm. A number of piers were found to be sitting on top of their cast-in-place concrete footings, and were easily knocked out by the excavator (Figure 9).

Under the western half of the floor plan, none of the piers were tied to the sub-floor structure. The wire hooks were attached to the piers, but were not connected to the bearers (Figure 9). The eastern half of the piers were connected with wire hooks to the bearers above.



Figure 9. Pier details, (left) upright piers showing wire hooks bent down and not used, (centre) pier cast only level to the ground, easily knocked out and (right) pier cast more competently into the ground, still encased in footing.

Ground:

The ground under the house footprint consisted of a thin, 200-300mm layer of topsoil, which was underlain by quite clean, silty sand. Excavation was only carried out to ~700mm depth, so below this depth the ground conditions are unknown. Above the topsoil there was sand ejecta up to 300mm thick over the western half of the floor plan (Figure 10).



Figure 10. Ground conditions, showing thin layer of darker topsoil, underlain by clean brown silty sand.

Summary of Damage

Surroundings

There was no paving on the section to indicate levels of ground heave and settlement and not much of this was evident on the ground.

The garage floor was totally covered with dried cakey sand ejecta up to ~200mm thick (Figure 11).

One set of steps on the North side was slightly separated (but still in tact) from the house by 5-10mm. It was also possible that the house has pulled away from the steps instead (Figure 12).

There was some damage to fences also with some sections leaning and ruptured.



Figure 11. P6: Liquefaction ejecta in the garage.



Figure 12. P7: Minor separation of concrete steps from house.

Structural Damage

Structural damage to the cladding inside and out was generally low-moderate throughout the majority of the house. Most of the windows outside had hairline cracks in the stucco at their bottom corners. There were a few larger vertical cracks of 2-3mm width in the external cladding, some of which were flexural along the bottom of the North wall (P9) and two of which were along the joints between the old walls and a new wall where a fireplace had been (P8). There were also 0-1mm separations around the two door/entrance frames (P10).

A number of doors inside swing open or closed by themselves due to the high tilt of the house, and some stuck when opened wide, due to local humping of the floor. However there was no obvious racking of any door frames.

Damage to the interior cladding was generally low, with only minor hairline cracks in the plaster in most rooms above windows and doors. There were a few more significant cracks in plasterboard and at plaster joints in some places, mostly situated in the central bedroom on the South side (Figure 14).



Figure 13. Exterior structural damage, P8: Cracks along joints between old wall and new wall over large concrete footing (old fireplace). P9: Flexural crack in stucco. P10: 1mm separation around doorframe.



P11 P12 P13
Figure 14. Interior structural damage. P11: Two floor-to-ceiling joint cracks in plasterboard. P12: More significant crack above window corner. P13: Minor crack at corner.

Foundation Damage

Foundation damage overall is high to severe throughout the building. There were a total of 24 cracks right around the perimeter, from hairline to 30mm width with 60mm horizontal offset. The majority of these cracks were 5mm wide or more. In both the East and West corners of the house, large sections (1m, 3m, 2m, 5m) of perimeter have broken completely from the rest and moved laterally out from under the building. In the case of the East corner, this section of perimeter has also settled more than the house above, causing significant structural damage (Figure 15).

There is also significant humping of the floor in a number of rooms. Some of this was made worse by the three large concrete footings, shown in Figure 6 and Figure 7, which did not appear to have settled as much as the rest of the house, causing large floor slopes.



P14 P15
Figure 15. Foundation damage, P14: low-moderate cracking in perimeter. P15: High-severe cracking and separation in perimeter.

Overall Building Deformation

Overall the building suffered severe differential settlement, with local break up and humping of the perimeter and floor causing some significant structural damage.

The overall differential settlement across the building was 200mm from West to East. A maximum differential settlement of 210mm was recorded, due to local humping in the highest room. Overall the tilting was uniform, but some local humping in rooms caused large slopes in other directions. There was an overall floor slope of 1.0° due to the differential settlement, but slopes of up to 2.2° locally in a variety of directions. This local humping caused some doors to stick on opening, and the large overall slope has caused some doors to swing open or shut by themselves.

Around the majority of the perimeter the building is protruding between 30 and 90mm over the concrete perimeter (Figure 18). It is assumed the house was built like this, although the large variation makes it difficult to tell. However, at the East and West corners of the house, sections of the perimeter wall are protruding from under the house. At the West corner (more minor) the corner has come out flush with the wall, suggesting a minimum of 30mm lateral movement (Figure 17). At the East corner, the perimeter is protruding by 40-60mm, suggesting an overall lateral movement of 70-150mm (Figure 16). From the East corner along the Northeast wall, the perimeter has also settled more than the house, from 0mm at the corner to 70mm lower 3m along the wall, leaving a gap under the wall. This gap closes up towards the front door steps (Figure 16). This long gap has caused the wall to sag, and produced at least 8 flexural cracks from the bottom of the wall, ranging from hairline to 2mm in width (Figure 13, P9).

As a result of this extra settlement along ~10m of the wall, the superstructure appears to be pushing hard into the perimeter at both the East and South corners, causing spalling of the concrete perimeter and stucco wall (Figure 16 and Figure 18). At the West corner, there is a 30mm gap between the perimeter and wall, suggesting that here the house has lifted up off the perimeter, due to the overall differential settlement (Figure 17). This shows that the relatively weak perimeter broke up and settled in a very broken and differential manner, with the separated superstructure of the house remaining intact and tilting overall along the perimeter, causing the range of gaps between house and perimeter wall.



Figure 16. P16: East corner showing spalling of stucco with protruding perimeter and gap under wall extending to front-door stairs.



Figure 17. P17: West corner showing gap under wall and protrusion of perimeter.



P18

Figure 18. P18: Southeast wall protruding over perimeter (like most of house).



P19

Figure 19: South corner spalling in perimeter.



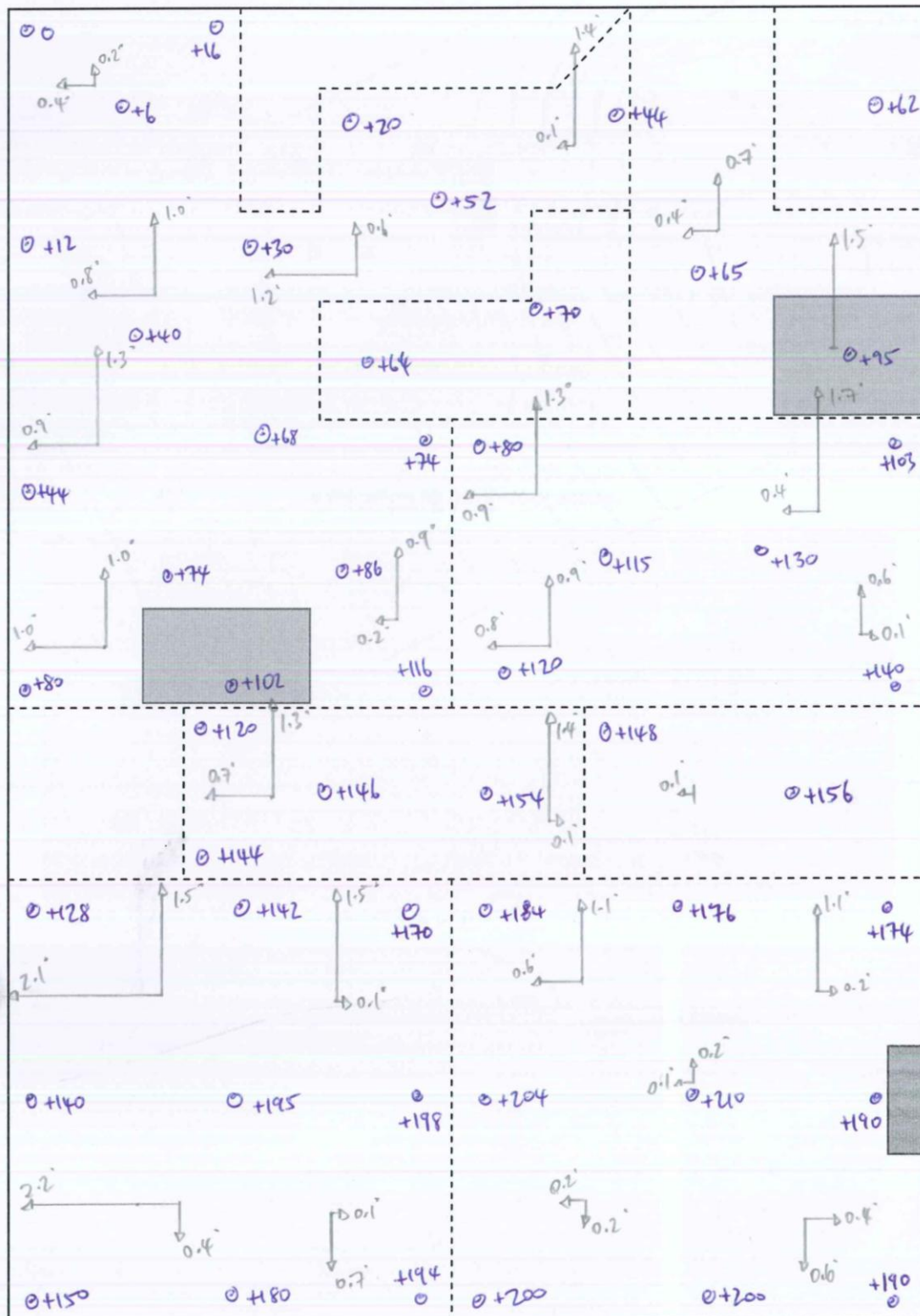


Figure 20. House plan layout showing room locations and floor slopes.

House ID:	House #4
Foundation Type:	Perimeter (brick) & Slab-on-grade
Number of Stories:	1
Age:	2000 (post-2000) – extension only
Plan Area:	Perimeter: 100m ² , Slab: 230m ²
Plan Shape:	Long rectangle/complex
Cladding	Brick with stucco
Roof	Iron



Figure 1. Overview of property, showing direction of overall photo views.



Figure 2. Local suburb, showing distance to nearby water.

Earthquake	Liquefaction	Lateral Shift (LiDAR)	Settlement (LiDAR)
September 2010	High	0.3-0.7m SE	0 to -0.3m
February 2011	High	0.3m S	-0.1 to -0.3m
June 2011	Mod.	0.1m W	0 to -0.2m
December 2011	-	-	0.1 to -0.1m



Figure 3. P1: View from E end of house, looking W at East half (slab) of house.



Figure 4. P2: View from halfway along house, looking W from East slab part towards perimeter section.



Figure 5. P3: View from NW end of house, looking SE at NW slab room and perimeter section behind.

Foundation Details

This large residential house had three different foundation types, based on different extensions. The original house consisted of a brick perimeter with short pier supports under the floor. Joined to this was a concrete slab at the same floor level, and on either side of these two were standard slab-on-grade floors (Figure 7).

Slab (Low)

The low slabs at both ends of the house had the same construction. Overall they were comparatively sturdy slabs (according to the contractor) compared to the 'average' house slab. Thick, with mesh reinforcing, a solid perimeter footing and decent ties between footing and slab, the slab was hard to lift and stayed intact well. No cracks were seen in the slab (except for one end where the footing was removed early) until it was lifted with the excavator, despite being driven all over with the excavator during demolition of the superstructure.



Figure 6. Low slab perimeter footing cross-section.

Perimeter Footing (Figure 6):

- 300mm (min) wide at the top, with the slab cast

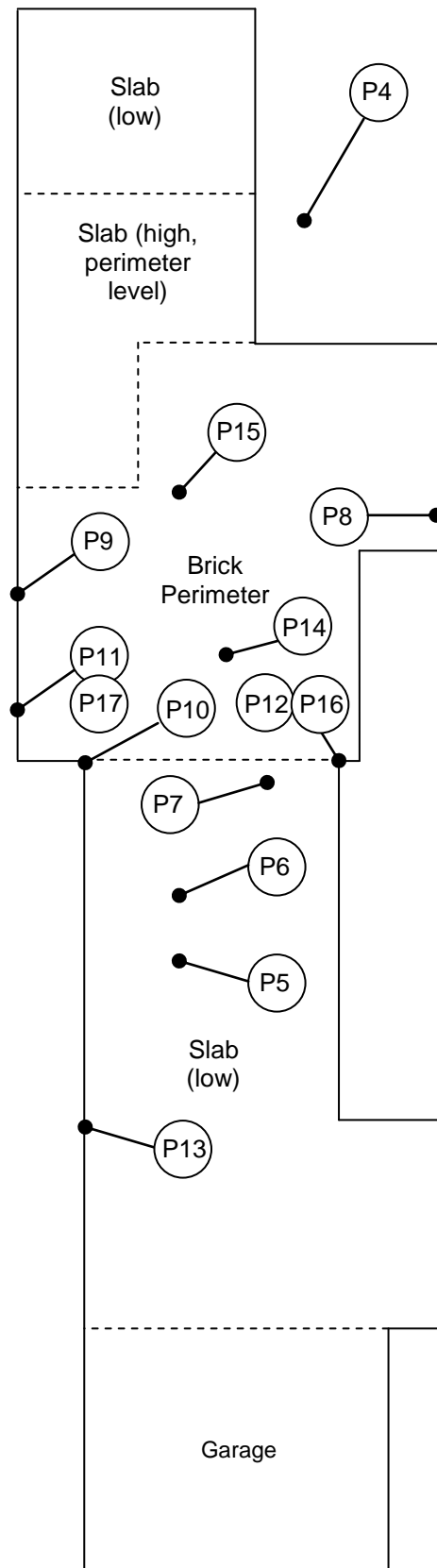


Figure 7. Floor plan showing general layout of house, with locations of following photos.

on top by at least 200mm. Total depth 500-600mm, with 100-150mm above the ground.

- Longitudinal reinforcing is 2x D16's, one 50-100mm from top, the other 50-100mm from bottom, both close to the centre of the footing.
- Tie bars into the slab are D10's at 600cc, extending out from the footing by 600-700mm.

Slab:

- 120-150mm thick over whole slab (better than 100mm standard minimum).
- Reinforced with 665 mesh (or equivalent), 5mm diameter, 150mm spacing. Positioned 10-50mm from bottom of slab.

Fill:

- Standard river run tailings. Rounded stone approximately 20-50mm diameter.

Slab (High)

The small raised slab area within the perimeter part of the building (Figure 7) was older than the two low slab areas, and of poorer construction. The slab was thinner and weaker and the fill was very poor.

Perimeter Footing:

- Total depth of 700-800mm, ~500mm above ground to top of slab, and 200-300mm into ground.
- Longitudinal reinforcing is (2 or 3x) R12 with no ties into slab.

Slab:

- 100-150mm thick (varies widely due to nature of fill). 665 mesh (or equivalent) reinforcing.

Fill (Figure 8):

- Floor as a whole sits on top of bare ground (perimeter footings extend into ground around the outside by 200-300mm).
- The footings go up to the floor level. The space under the slab is filled with loose bricks (whole, broken and some plastered together) and tailings up to 200-300mm in diameter. The concrete was then poured on top of this, filling some of the voids between the loose fill, until the floor level was reached.
- The floor was built over some un-altered structures that were already present (i.e. old concrete stairs and patio area and brick foundation walls).



Figure 8. (left) high slab overlaying mixed fill, (right) size and material range of mixed fill.

Perimeter

The perimeter section of the house was a lot smaller than the total slab area. It had a relatively weak solid brick perimeter wall which supported the full brick cladding, and quite solid concrete piers which were tied to the bearers with wire hooks.

Perimeter Wall:

- 200mm of bricks in the ground, with 500-600mm to the floor level. Bricks were laid continuously up the full height of the wall. Perimeter wall appeared in no way attached to the rest of the foundation or interior floor structure.

Piers (Figure 9):

- Piers extended ~250mm into the ground, with 500mm above the ground to the underside of the bearers.
- Base 250-300mm square, top ~170mm square.



Figure 9. Piers attached to bearers, showing brick perimeter surrounding (left), and exposed pier (right).

Summary of Damage

Surroundings

No retaining walls were present at the site. There was little cracking in the paving, but significant settlement in all. The concrete path along the back (Southwest side) of the house was tilting between 4.3-6.0° towards the house. Paving at the north corner of the house was tilting between 0.9-1.6°, with maximum settlement of 112mm towards the house (Figure 10). And concrete paving halfway along the Northeast side of the house was tilting between 0.5-1.5° to the south, with consistent settlement of 20mm towards the house, and 204mm overall differential settlement to the south. Neither of these two areas of paving were connected structurally to the slab foundations they were adjacent to. Much of the fencing was leaning and separating. There was some evidence of old liquefaction damage around the property, with some possible cracks and damage to the lawn. However this was quite overgrown and not obvious. No sand ejecta was present.



Figure 10. P4: North corner paving, showing direction of tilt in towards house.

Structural Damage

There was significant structural damage to both parts (perimeter and slab) of the building. A number of rooms in the Southeast (low slab) section of the house had only minor interior damage, with minor vertical cracks in the plasterboard above windows and doors. However there were also a number of doors and windows in the Southeast low slab part of the house which showed the effects of significant racking (Figure 11). In the older perimeter and raised slab part of the house, there was also significant structural damage due to the foundation movement. Significant racking of door frames, and numerous minor to major cracks in the plasterboard were seen. There were two areas in the house that suffered buckling and pop-out of an entire wall of plasterboard. These were the South wall of the garage and the Southeast side of the wall between the perimeter and low slab sections in the centre of the house (Figure 12, P7). Exterior damage included cracking in the brickwork cladding, from 1 to 10mm wide, in the horizontal, vertical and diagonal directions. Most of the minor cracks were located at door/window corners all around the house, but to a lesser extent in the Southeast low slab half of the house than in the central perimeter part (Figure 12, P8). The most significant structural damage occurred at the interface between the slab and perimeter sections of the building in the middle of the house. The two parts of the house were basically ripped apart, causing a 50-100mm separation between interior plasterboard walls on the south side, with a large diagonal crack in the brick work on the corresponding exterior wall. On the north side, the walls around the corner of the perimeter section had separated by 10-20mm from the foundation. This also caused the interior walls to pull away from the ceiling in some places near the junction (Figure 12, P9-P12). There was a horizontal crack running the length of the Southeast low slab half of the house between the slab and wall above. In some locations the wall was pushing out over the slab by up to 10mm (Figure 12, P13).



P5



P6

Figure 11. P5: Vertical cracking above door frame. P6: Significant racking of door frame.



Figure 12. P7: Buckling plasterboard. P8: Minor external cracking. P9: Major diagonal crack in brickwork. P10: South wall exterior structural separation. P11: South wall interior separation. P12: North wall exterior separation. P13: Crack between SE low slab foundation and wall.

Foundation Damage

Foundation damage overall was severe, and was caused by differential settlement, hogging and dishing and humping of various parts of the floor.

There were no visible cracks in the Southeast low slab half of the house, despite severe differential settlement of 232mm over the length of the floor to the Southeast, and multiple instances of hogging and dishing over this length, evidenced by the changes in floor slope angles over the length. The slab did not crack until it was lifted up with the excavator at the end of demolition, despite being repeatedly driven over with the excavator during demolition of the superstructure. Cracking was caused at one end of the slab by the excavator, but this was because a section of the perimeter footing under it had been removed, along with some of the supporting river-run tailings fill.

There was also no cracking visible in the low slab room at the Northwest end of the house, as this only had differential settlement of 10-15mm across the floor, with small slope angles of 0-0.2°.

There was significant damage in the raised-floor part of the house. The perimeter part of this area had floor humping in most rooms, shown by the differences in slope angles measured. This was found to be caused by sand ejecta around the piers supporting these floors, which caused pushing up and sinking of the piers. The floor and pier structure was unconnected to the brick perimeter and wall part of the house, which caused interior separation between the walls and floor of 10-20mm in the E corner of the perimeter section, as the floor sank and the walls moved independently of this. There were three significant cracks in the tiling of the raised floor, two along the edges of the raised slab, caused by a difference in slope in the perimeter, pier-supported and slab-supported parts of the floor, and one between two sections of the raised slab, which were separated by a continued brick foundation wall extending through the slab from the perimeter section.



Figure 13. P14: Sand boils around piles causing humping of floor.



P15



P16



P17

Figure 14. P15: Crack in floor tiles along slab joint. P16: Separation between timber floor and wall (NE side). P17: Separation between floor and wall (SW side)

Overall Building Deformation

There was severe overall deformation to the structure, with significant differential settlement in multiple directions, racking, hogging and dishing of the floor and superstructure, and rip-apart of the two halves of the building.

The Southeast half of the house (low slab) was tilting severely to the Southeast, away from the Northwest half of the house. Overall differential settlement from the interface between the two parts of the house is 232mm to the Southeast. More locally, this half of the house is in two quarters which are both dishing. This has caused high levels of racking in windows and doors along the main axis of the house in (Northwest quarter). And buckling of plasterboard wall, mirrored by cracking in the external brickwork along the South wall of the garage. This dishing is shown by the slope angles. The slab slopes at 1.3-1.4° next to the perimeter interface, which reduces to ~0.5° halfway along the Southeast slab half of the house indicating dishing between these two. The slopes then increase again to 0.7/0.8°, and the garage is actually tilting back towards the rest of the house by 0.2-0.4° showing there is dishing between these two also. This means that there is hogging between the two areas of dishing, halfway along the Southeast slab section. The differential settlement of this half of the house is also seen in the concrete patio on the North side of this low slab section of the house, which has overall differential settlement of 186mm also to the Southeast.

The central part of the house (perimeter foundation and raised slab) is tilting directly West, and away from the Southeast low slab section. Overall this section of the house has a differential settlement of 172mm to the West. There is floor humping throughout the perimeter part of the house, again indicated by the slope angles measured in each room. Sand boils were identified under the areas where humping was observed, suggesting piles had been pushed up by the ejection of sand and water. The raised slab section of this part of the house was tilting more to the Southwest than the perimeter part, causing cracking in the floor tiles along the line where the slab started.

The tilting in different directions of the two halves of the house resulted in large structural damage around the interface between the two, as discussed in the structural damage section.

The low slab room at the NW end of the house only suffered minor tilting between 10-15mm across the room, with only minor slope angles between 0-0.2° measured.

The crack between the slab/wall interface along the Southeast wall of the low slab half of the house suggests that the whole wall may have buckled outwards, due to the dishing in this section of the house.

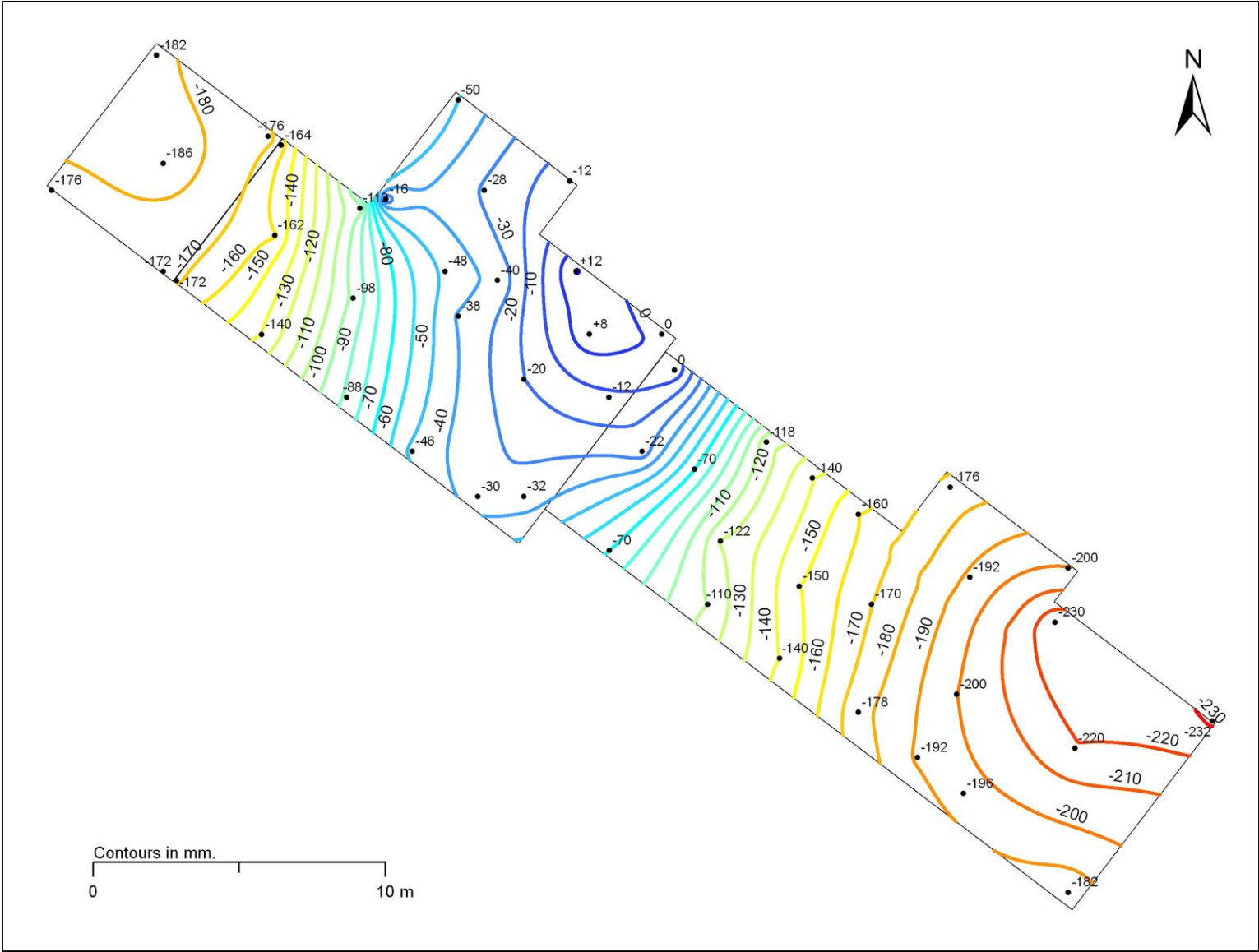




Figure 16. House plan layout showing room locations and floor slopes.



House ID:	House #5
Foundation Type:	Slab-on-grade
Number of Stories:	2
Age:	1990 (1980-99)
Plan Area:	100-200m ²
Plan Shape:	“L”
Cladding	Brick veneer, Iron
Roof	Iron



Figure 1. Property details, (left) overview of property, showing locations of overall photos and (right) distance to adjacent river.

Earthquake	Liquefaction	Lateral Shift (LiDAR)	Settlement (LiDAR)
September 2010	High	0.4m NE	0 to -0.2m
February 2011	High	0.15m SE	-0.1 to -0.3m
June 2011	Mod.	0.15m NW	-0.1 to -0.2m
December 2011	-	0.1m N	-0.1 to -0.2m



Figure 2. P1: View of house (partially demolished) from NW.



Figure 3. P2: View of house (partially demolished) from SE.

Foundation Details

Perimeter Footing:

The perimeter footing ranged between 200-260mm wide and was 700-900mm deep. It had longitudinal reinforcement consisting of 2x D10's in the bottom and 1x D16 in the top and R5 stirrups around these. It was strong in comparison to the standard for residential slab-on-grade foundations, according to the contractor lifting it up.

Slab:

The slab ranged from 95-160mm in thickness and was reinforced with wire mesh, which was positioned anywhere from the bottom 20mm to near the middle. The extra thickness in much of the slab was assumed to be the reason that very few cracks were seen in the floor, despite the high degree of differential settlement across the floor plan of the building.

Fill:

The fill beneath the slab was river run tailings, round stone between 20-50mm in diameter.



Figure 4. Foundation details, (left) perimeter footing in ground showing fill, (right) concrete slab showing reinforcing mesh.

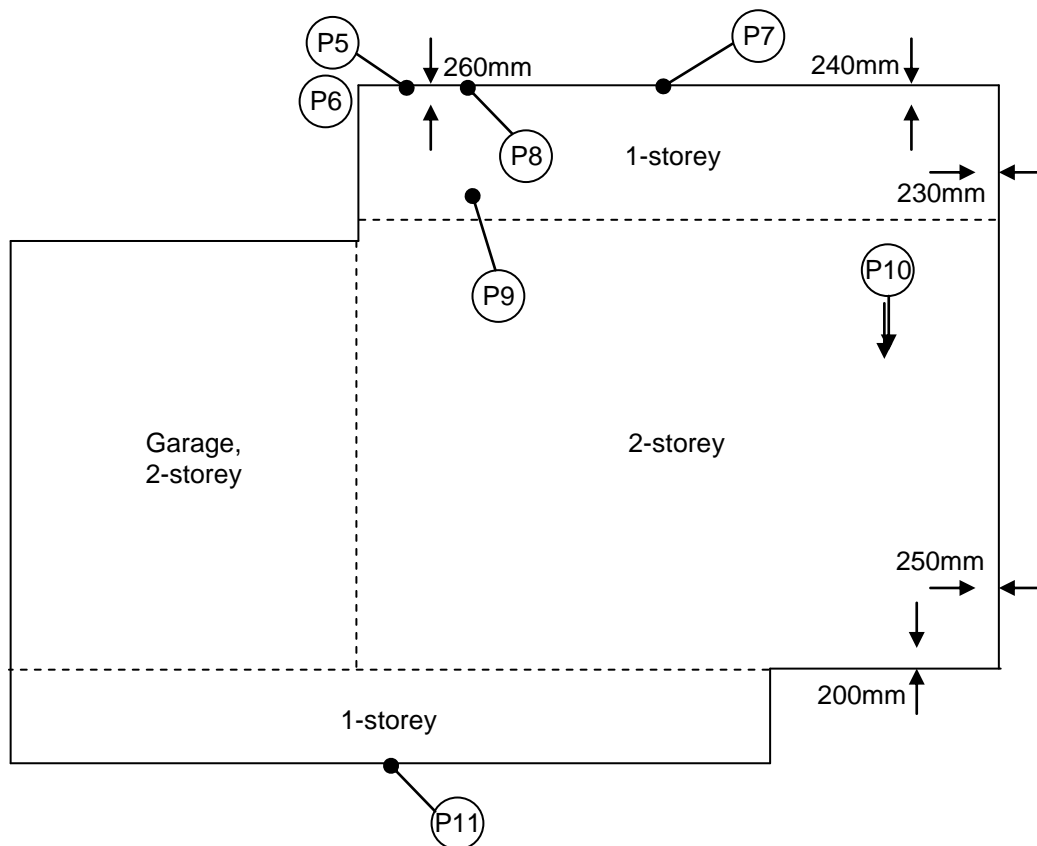


Figure 5. Floor plan showing general layout of house, with perimeter footing widths and locations of following photos.

Summary of Damage

Surroundings

No decks, steps or retaining walls were present at the site. All concrete paving was severely damaged, with most of the paving cracked and separating by up to 50-70mm (Figure 6). Tilting and settlement of the pavement was also high. There was little to no damage to fences.



Figure 6. Cracking and separation in the paving around the house, P3 (left) and P4 (right).

Structural Damage

Damage to the overall superstructure was generally low.

The interior cladding had already been removed when the inspection was conducted. But from talking to the contractor who was doing the demolition, the damage to the interior was minor. There were minor cracks in the plaster above windows/doors. There were no windows or doors that were jamming.

The external cladding also suffered only minor damage. There was only minor cracking at window corners in the external brickwork (Figure 7).



P5



P6



P7



P8

Figure 7. Damage to the external brickwork, showing minor cracking at window corners.

Foundation Damage

Visible damage to the exposed floor slab was low. There was only one minor crack in the slab, around 0-1mm wide and 3-4m long running E-W along the slab in the middle of the North side of the house (Figure 8). Tilting of the slab was high, between 48-102mm across the building, sloping down to the North. The North half of the house was more severely tilted, with slopes between 0.5°-1.7° to the North, compared to the South half of the house with slopes between 0.1°-0.3° to the North. This indicates hogging along an E-W line somewhere around the middle of the house, and is assumed to be the reason for the crack in the floor slab. This crack was also situated at a change in plan from 2-storeys to 1-storey in height.



Figure 8. P9: Crack in concrete slab.



Figure 9. P10: View of slab in Kitchen/Dining area from the North.

Overall Building Deformation

The building had an overall differential settlement of 48-102mm from the South side down towards the North. The 1-storey section of the building on the North side settled slightly more than the rest of the building, causing low-moderate hogging of the overall structure. This was indicated by different slopes in the floor between 1 and 2 storey sections, and the minor crack in the slab.

The building had an overall global settlement of 60-70mm on the South side of the house (the high side), which was shown by a vertical offset between a sewerage pipe entering the slab (Figure 10). Adjusting for the tilt of the house, this corresponds to an overall global settlement of 110-120mm.



Figure 10. P11: Vertical offset of sewerage pipe between ground (left) and house (right).

Horizontal separation between the house and paving on the south side of the house was found to add to a total of between 70-80mm. This suggests that lateral spreading has occurred on the south side of the house towards the river 55m to the south. So overall the house has experienced lateral spreading towards the river to the South, and has settled globally 110-120mm, and differentially by up to 102mm back from the river towards the North.

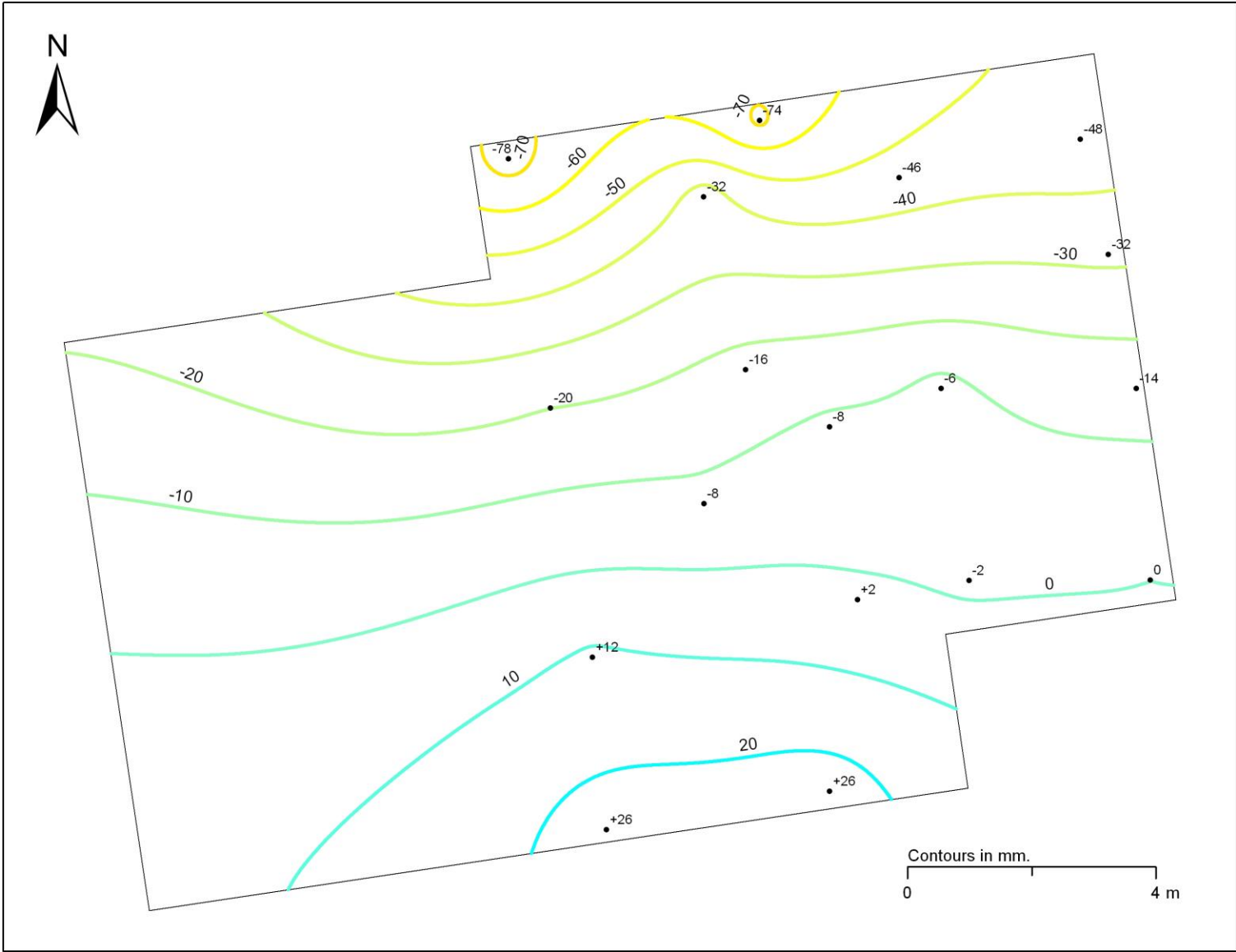


Figure 11. Floor plan showing floor levels and contours.

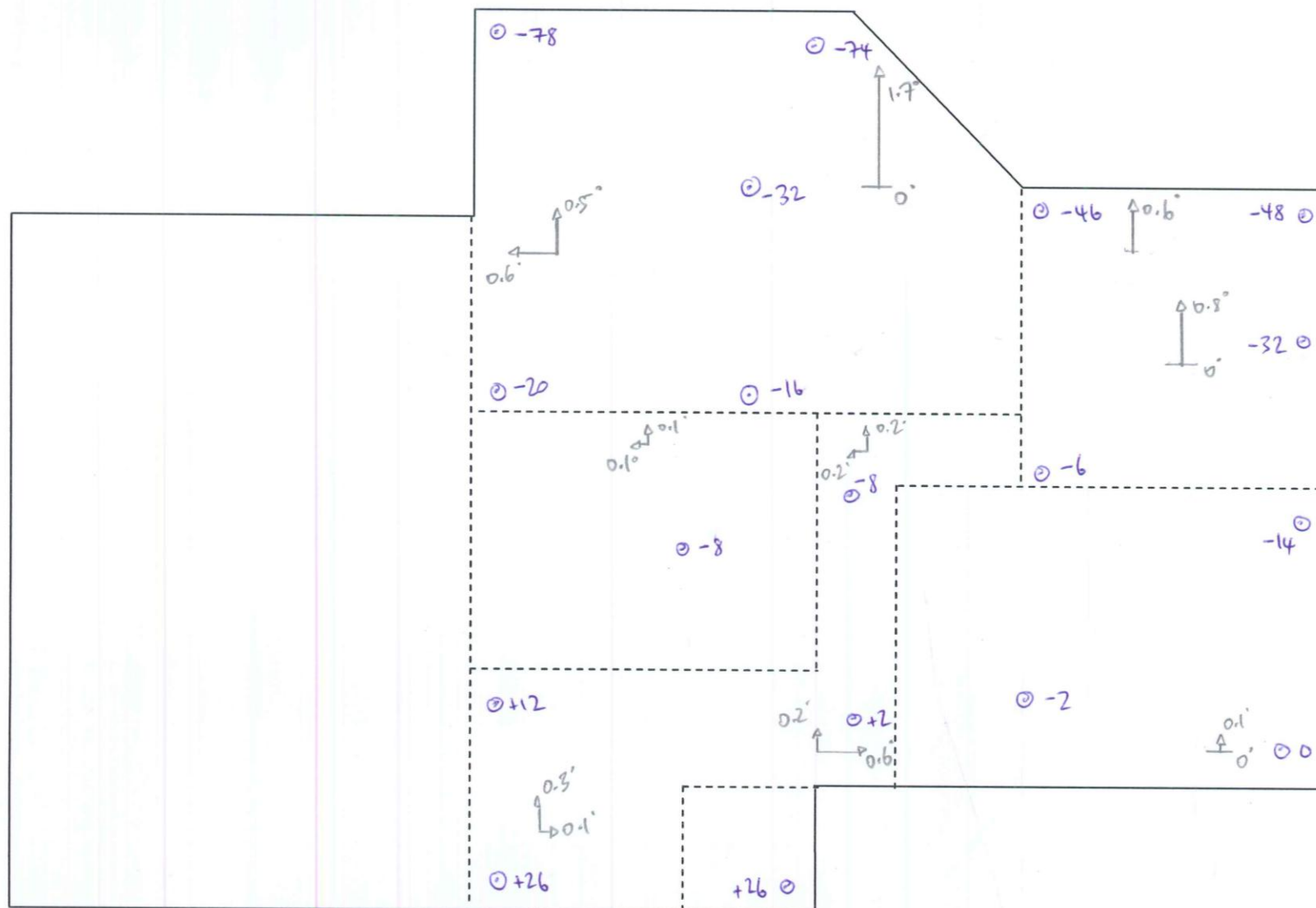


Figure 12. House plan layout showing room locations and floor slopes.

House ID:	House #6
Foundation Type:	Slab-on-grade
Number of Stories:	1
Age:	1980-99
Plan Area:	110m ²
Plan Shape:	"T"
Cladding	Weatherboard, Stone veneer
Roof	Iron



Figure 1. Property details, (left) overview of property, showing direction of overall photo views and (right) distance to adjacent river.

Earthquake	Liquefaction	Lateral Shift (LiDAR)	Settlement (LiDAR)
September 2010	High	0.8m SSE	0 to -0,2m
February 2011	Severe	0.45m S	-0.1 to -0.3m
June 2011	High	0.1m W	0.1 to -0.1m
December 2011	Low	-	0.1 to -0.1m

Sand ejecta and liquid came into the house. Very high/severe liquefaction three times, and less in numerous other large aftershocks.



Figure 2. P1: View of house from Gayhurst Rd, looking SE.



Figure 3. P2: View from rear of house, looking SW.

Foundation Details

Footing:

- D10 reinforcing bars
- Tied into slab

Slab:

- Ranges from 95-160mm thick
- Reinforced with mesh, but not 665 (weaker)
- A lot of round aggregate in concrete, slab broke apart easily

Fill:

- River-run round stone fill
- 20-50mm diameter

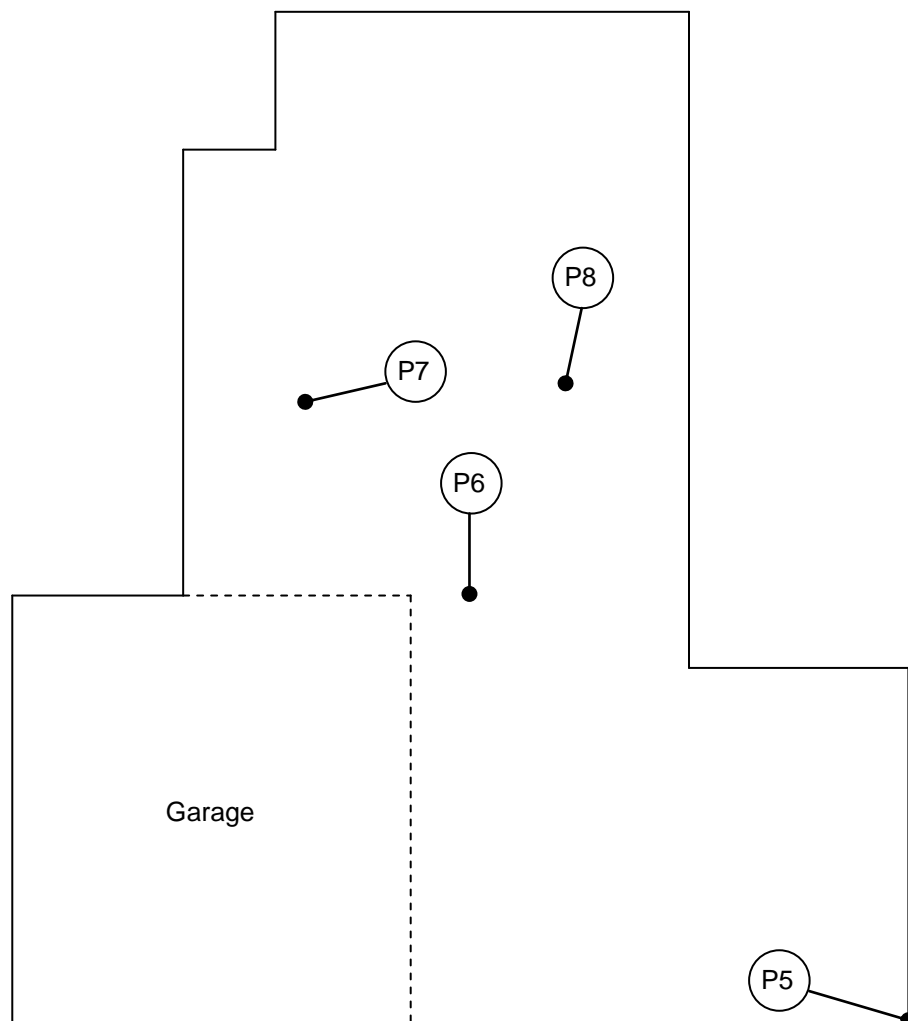


Figure 4. Floor plan showing general layout of house, with locations of following photos.

Summary of Damage

Surroundings

No decks, steps or retaining walls were present at the site. Concrete paving was severely damaged, with most of the paving cracked and separated, with significant heaving and settlement (Figure 5 and Figure 6). There was little to no damage to fences.



Figure 5. P3: Separation of concrete paving.



Figure 6. P4: Cracking in asphalt paving.

Structural Damage

There were low levels of damage to the external cladding, with some smaller cracks in the brick cladding and a couple of places where the corner bricks at ground level were falling off (Figure 7). The interior also suffered relatively minor damage, with some small vertical cracks above doors and windows and a few in corners (Figure 8). Despite the low level of damage to the cladding, there was significant structural damage overall, as most doors and windows were sticking, including both garage doors which were jammed and would not open.



Figure 7. P5: Cracking and separation of bricks at corner of house, overall (left) and close-up (right).



Figure 8. P6: Minor vertical cracking above door frame and at wall/ceiling interface in interior plasterboard.

Foundation Damage

There was extensive minor cracking right across the foundation slab. Cracks were between 0-2mm wide and 2-5m long in multiple directions and intersecting with each other (Figure 9). There was extensive settlement and tilting of the house, with up to 180mm differential settlement over the length of the house. The foundation in the North wing of the house suffered some hogging damage in the N-S direction, with local differential tilting in other parts of the foundation. Floor slopes up to 1.0° were found. The extreme tilting and extensive differential slopes of the foundation are assumed to be due to the severe liquefaction suffered and the weak construction of the foundation with round aggregate.



Figure 9. Minor cracking in concrete floor slab, P7 (left) and P8 (right)

Overall Building Deformation

The building had overall global settlement of 130-170mm, which was measured from the sloping lawn that was once flat (after discussion with the owners). In addition, the whole house is tilting differentially between 74mm and 118mm to the East. The North part of the house is also hogging in the N-S direction, and the south part of the house is tilting quite uniformly towards the north part of the house by up to 0.7°. Despite the large movement of the structure, damage to the superstructure is relatively minor, suggesting the construction of the superstructure was very competent. The house is located in the centre of a large (approximately 350m diameter) semi-circular bend in the nearby river Avon, so did not experience a large degree of lateral spreading under the footprint of the house, although significant lateral spreading did occur in all directions closer to the river. However, the severe liquefaction under the house and the weak construction of the foundation caused the large structural damage and overall settlement of the building.

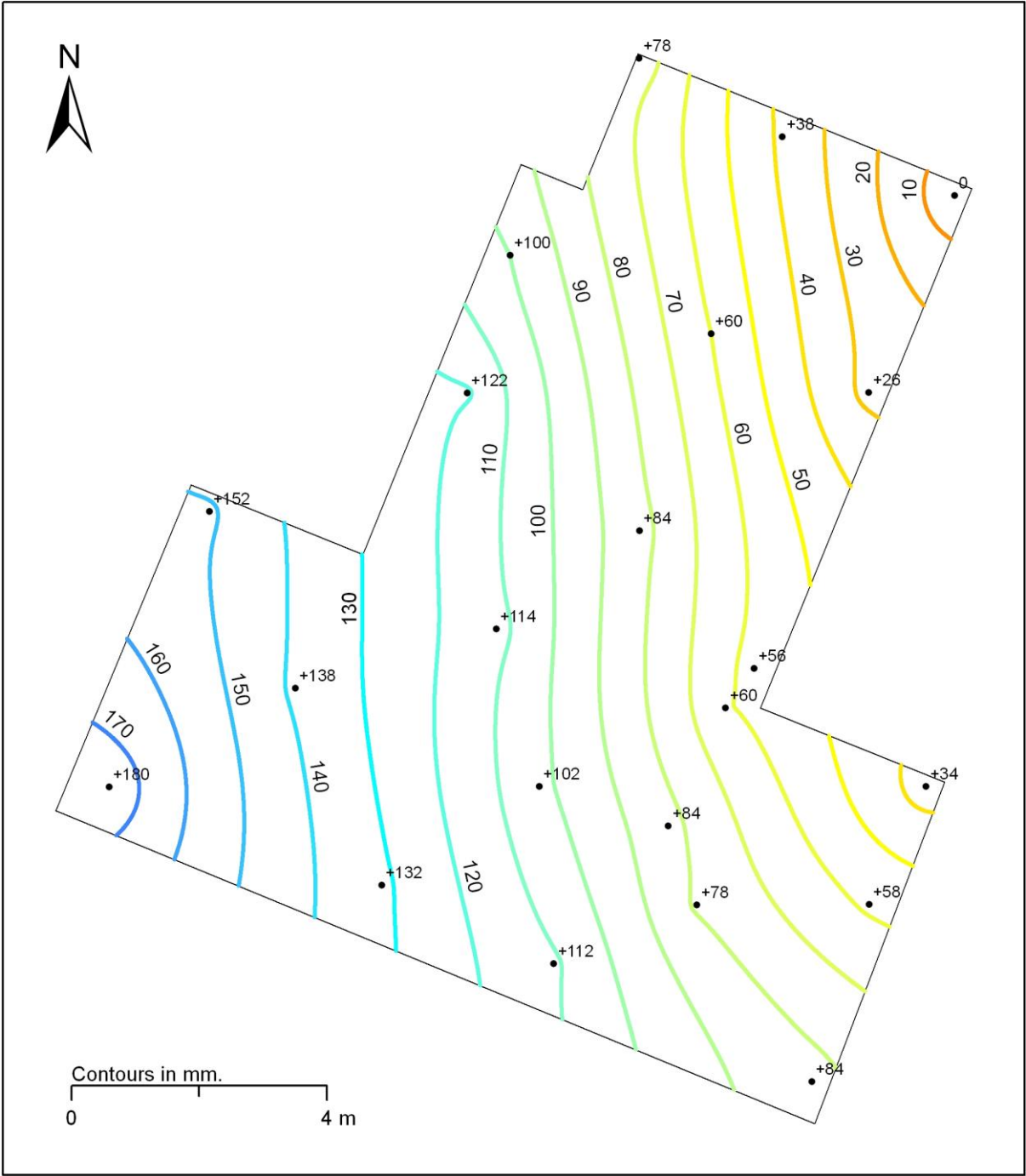


Figure 10. Floor plan showing floor levels and contours.

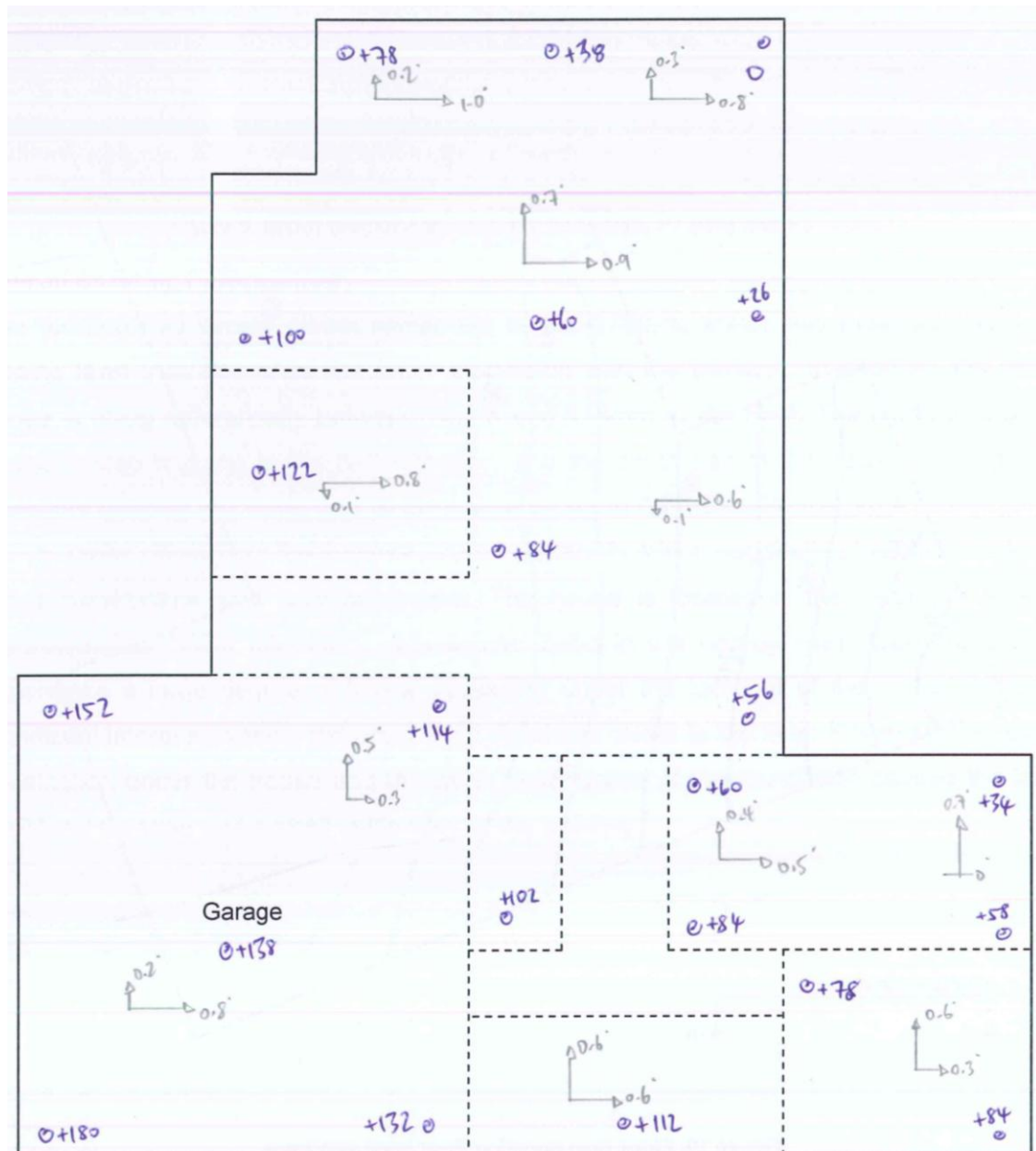


Figure 11. House plan layout showing room locations and floor slopes.

House ID:	House #7
Foundation Type:	Slab
Number of Stories:	1
Age:	1953/54
Plan Area:	130m ²
Plan Shape:	Rectangle/Complex
Cladding	Stucco & Concrete Block
Roof	Concrete Tiles



Figure 1. Overview of property, showing direction of overall photo views.



Figure 2. Local suburb, showing distance to nearby water.

Earthquake	Liquefaction	Lateral Shift (LiDAR)	Settlement (LiDAR)
September 2010	Minor	0.25m E	0 to -0.2m
February 2011	Mod-Major	0.2m SW	-0.1 to -0.4m
June 2011	Mod.	0.1m NW	0.1 to -0.1m
December 2011	Mod-Major	-	0.1 to -0.2m



Figure 3. P1: Northwest side of the house.



Figure 4. P2: West end of the house.



Figure 5. P3: Southeast end of the house.

Foundation Details

Footings:

In addition to the standard perimeter foundation walls expected under a standard slab-on-grade foundation, this house also had intermediate foundation walls under almost every interior wall. These provided significant extra strength and stiffness to the foundation.

Perimeter footing (Figure 7)

- 300mm wide around the whole perimeter
- 550-700mm deep
- Longitudinal reinforcement 3x R12's, 2 in bottom, 1 in top
- No visible transverse reinforcement

Intermediate foundation beams (Figure 7)

- 150mm wide
- 500mm deep
- Under every significant load-bearing wall within house footprint.
- Ranged from longitudinal reinforcement of 1xR12 in bottom, to 3xR12's, as for perimeter footing.

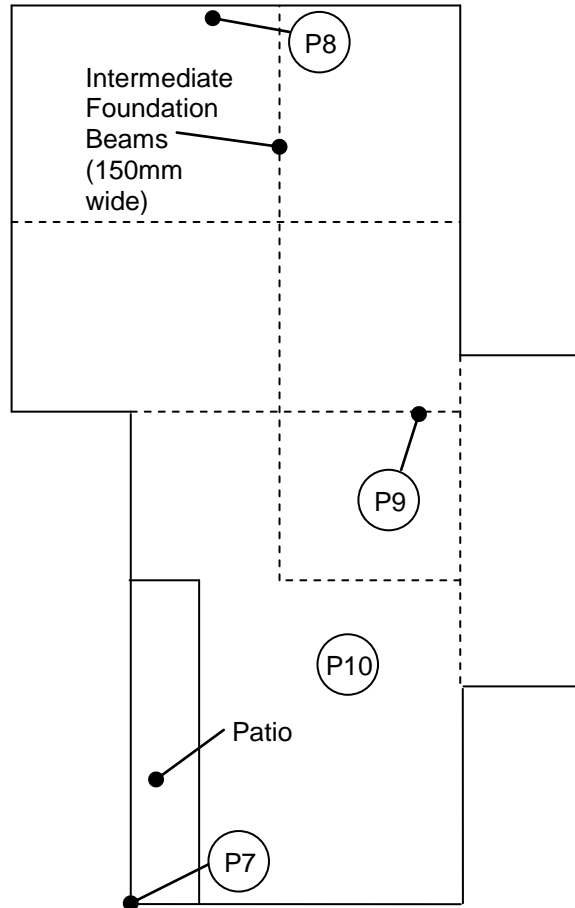


Figure 6. Floor plan showing structural layout of foundation and locations of following photos.



Figure 7. Footing details, (left) perimeter footing with slab on top and fill inside and (right) intermediate foundation beam.

Slab:

The concrete slab floor was un-reinforced, but constructed of well-made concrete with well-graded aggregate. There were some round pieces of aggregate, but the slab stayed together very well when being lifted with the excavator. The slab was ~100mm thick everywhere. It was also covered in ~50mm of additional composite covering (Figure 8). This consisted of a wood-chip/cement combination and was assumed to be for insulation.



Figure 8. Slab details, (left) composite covering and (right) competent fill with 100mm thick slab on top.

Fill:

The fill under 90% of the slab was very competent, uniformly graded angular gravel, suspected to be either AP40, AP60 or equivalent (Figure 8). The fill under the patio on the North side (Figure 6) is (volcanic) stone, 100-300mm in diameter.



Figure 9. Volcanic stone fill, attached to bottom of an uplifted section of concrete floor slab.

Summary of Damage

Surroundings

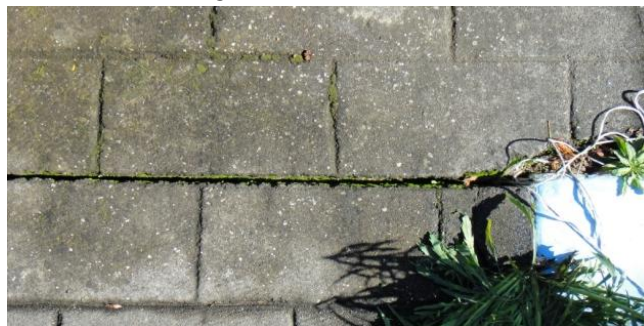
There was moderate damage to the surrounding property. Decks showed some low tilting and separation, though this may have been due to age rather than the earthquake. Fences were tilting in a few places. Concrete paving was cracked in most locations, along joints in the concrete. With separation up to 20mm between sections. The pool was tilting 0.5-0.7° to the N and separated from the surrounding paving by 10-20mm in some places.



P4



P5

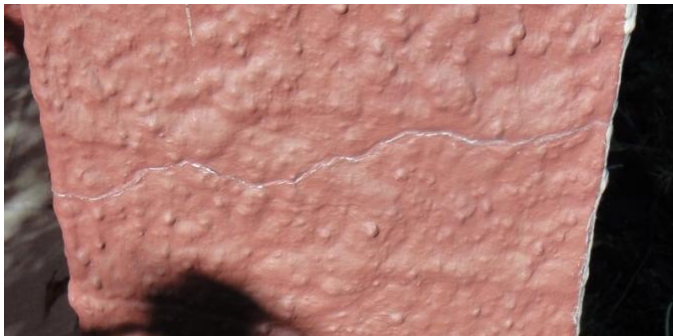


P6

Figure 10. Damage to surroundings. P4: Pool area tilting to N (left). P5: Separation in paving around pool. P6: Cracking and separation in paving.

Structural Damage

Overall the structural damage to the house was very low. There were three hairline cracks in the external stucco cladding. In the interior there were two areas of significant cracking identified. The concrete wall behind where the fridge was in the kitchen had numerous horizontal, vertical and diagonal cracks from 0-2mm wide. The wall in one of the bedroom cupboards also had a 1-2mm diagonal crack. There was cracking in the plaster ceilings in every room in the house from 0-2mm wide running all ways. Except the kitchen which had a standard plasterboard ceiling.



P7



P8



P9



P10

Figure 11. Structural damage. P7: Hairline crack in stucco cladding. P8: Cracking in bedroom cupboard. P9: Cracking in concrete wall behind fridge. P10: Cracking in plaster ceiling.

Foundation Damage

Foundation damage was low overall. There was a maximum of 28mm of differential settlement along the length of the house, for an overall slope of 0.1° . The maximum tilt measured was 0.4° found in two locations.

Overall Building Deformation

The building had low damage overall, with minimal settlement. Local distortion of the floor was also low, with only two tilt measurements of 0.4° taken, the rest being 0° , 0.1° or 0.2° . The overall differential settlement was 28mm over the length of the house, too small to say the house has settled in any direction. There was no evidence of global settlement of the house relative to the surrounding land.

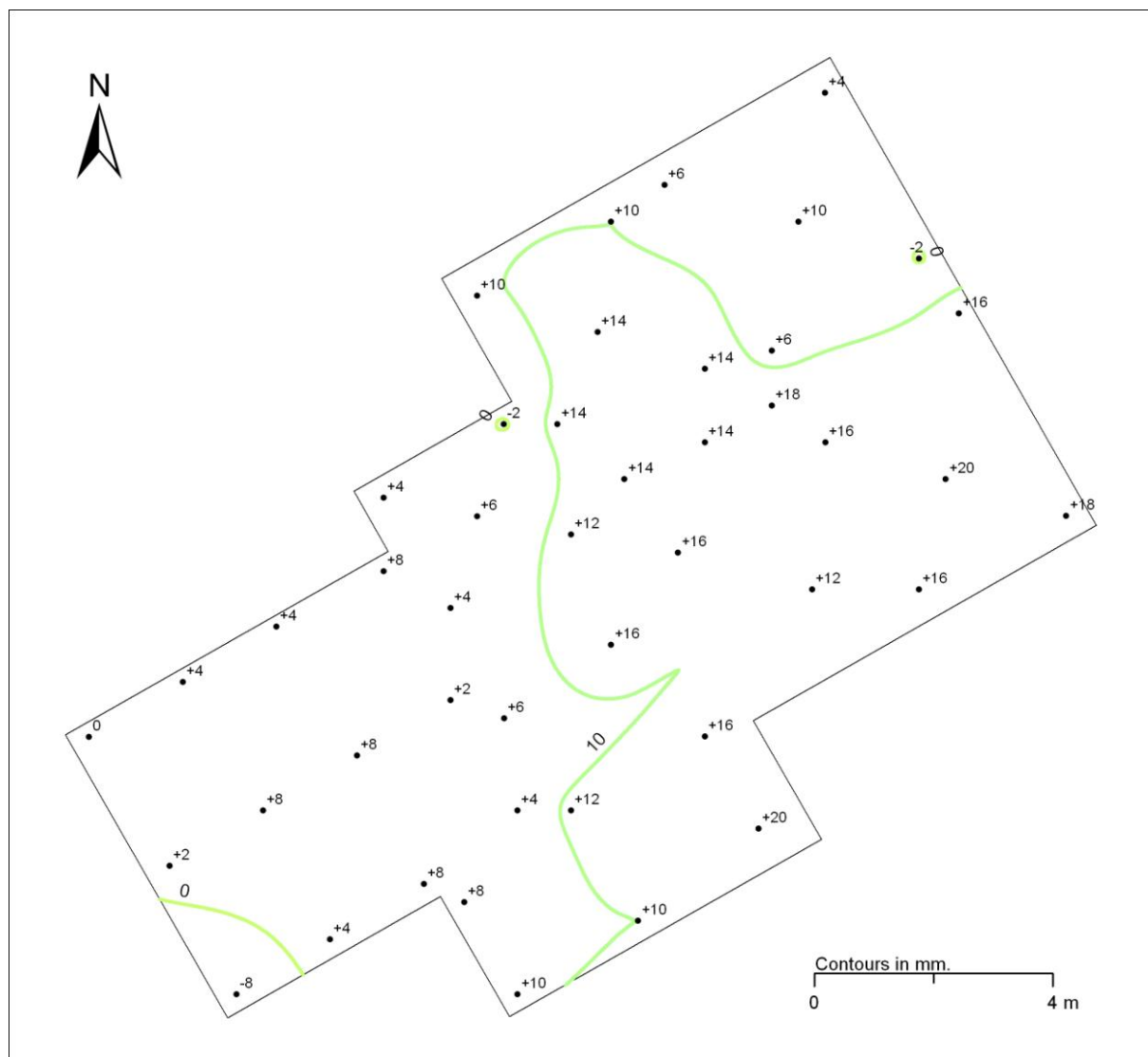


Figure 12. Floor plan showing floor levels and contours.

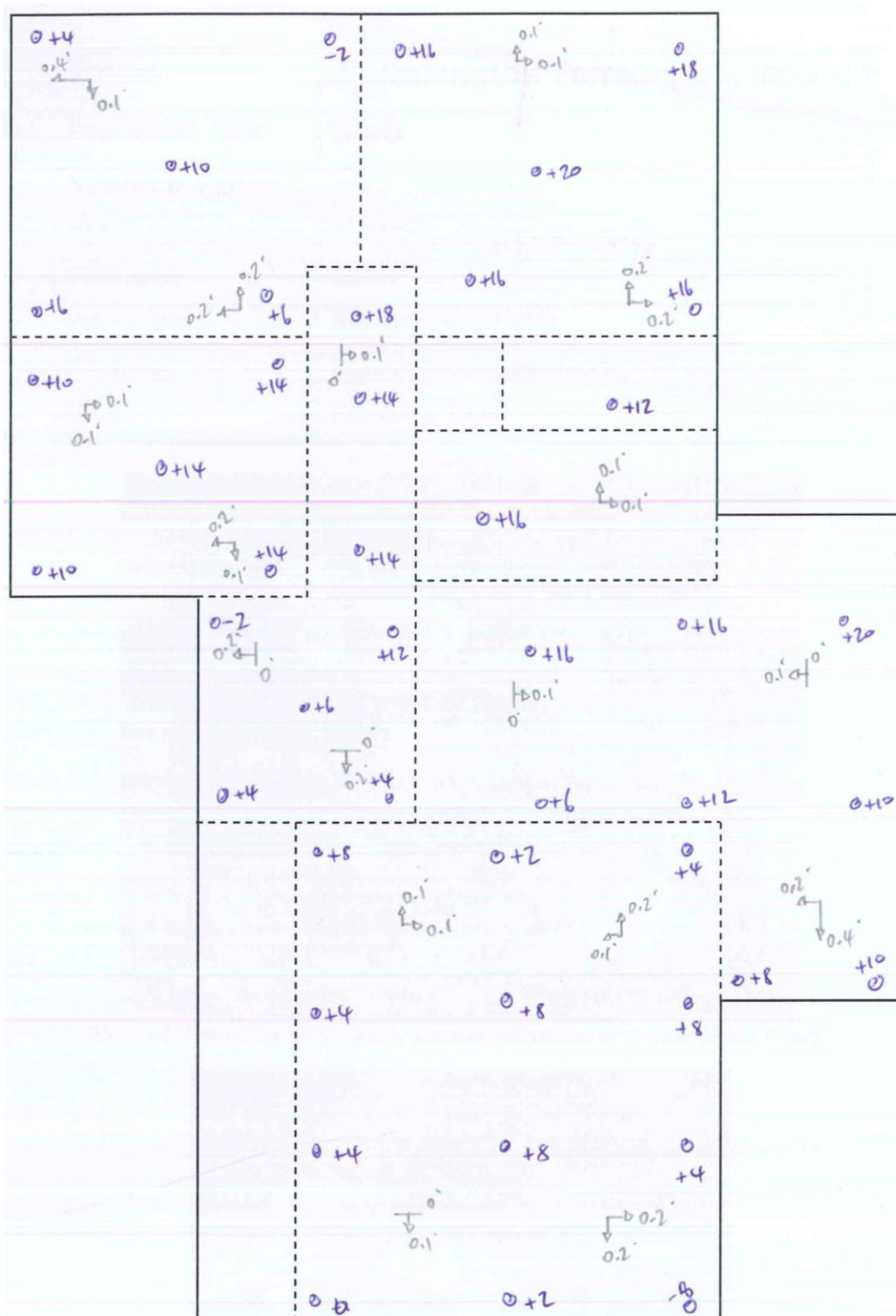


Figure 13. House plan layout showing room locations and floor slopes.